U.S. DEPARTMENT OF COMMERCE
National Technical Information Service
PB80-115801

Seventh Symposium on Advanced Treatment of Biologically Treated Effluents Including Nutrients Removal, USA/USSR Held at Moscow, USSR on November 12-13, 1978

(U.S.) Environmental Protection Agency, Washington, DC

Nov 79

United States
Environmental Protection
Agency

Office of
Water Program Operations
Washington DC 20460

November 1979

PB80-115801

EPA

Symposium on Advanced Treatment of Biologically Treated Effluents Including Nutrients Removal

USA·USSR

U.S.A. – U.S.S.R.
Working Group
on the Prevention of
Water Pollution
from Municipal and
Industrial Sources

November 12-13, 1978

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SPRINGFIELD, VA. 22161

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO.

2.

3. RECIPIENT'S ACCESSION NO.
PB80-115801

4. TITLE AND SUBTITLE
USA/USSR Symposium on Advanced Treatment of Biologically Treated Effluents Including Nutrients Removal - Seventh in Series

5. REPORT DATE
November 1979

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)

8. PERFORMING ORGANIZATION REPORT NO.

9. PERFORMING ORGANIZATION NAME AND ADDRESS
Office of Water Program Operations
Municipal Construction Division (WH-547)
Washington, D.C. 20460

10. PROGRAM ELEMENT NO.
2BG647

11. CONTRACT/GRANT NO.

12. SPONSORING AGENCY NAME AND ADDRESS
USEPA
Office of Water Program Operations
401 "M" Street, S.W.
Washington, D.C. 20460

13. TYPE OF REPORT AND PERIOD COVERED

14. SPONSORING AGENCY CODE
EPA/700/02

15. SUPPLEMENTARY NOTES

16. ABSTRACT

The seventh cooperative US/USSR symposium on the "Advanced Treatment of Biologically Treated Effluents, Including Nutrients Removal" was held in the Soviet Union at the Moscow headquarters of Gosstroy on November 12th and 13th, 1978. This symposium was conducted in accord with the sixth session of the Joint USA/USSR Commission held in Moscow, USSR from November 14 through 18, 1977.

The twelve papers that were presented at the symposium (six US and six USSR) are reprinted in English in this volume in accord with the protocol signed by the delegation leaders on November 24, 1978.

Some of the [illegible] [illegible] discussed in the report are
[illegible] [illegible] treatment, [illegible] filtration
[illegible] [illegible] water [illegible] and land
application of wastewater

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Water Pollution Control	Advanced treatment of Effluent Nutrients removal by land treatment	

18. DISTRIBUTION STATEMENT	19. SECURITY CLASS (This Report)	21. NO. OF PAGES
	Unclassified	99
	20. SECURITY CLASS (This page)	22. PRICE
	Unclassified	A05/A01

EPA Form 2220-1 (9-73)

i

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Preface

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This symposium was sponsored under the auspices of the Working Group on the Prevention of Water Pollution from Municipal and Industrial Sources. The cochairmen of the Working Group are H. P. Cahill, Jr. of the United States Environmental Protection Agency and S. V. Yakovlev of the Department of Vodgeo in the Soviet Union.

The United States delegation was led by Harold P. Cahill, Jr., Director Municipal Construction Division, U.S. Environmental Protection Agency.

The twelve papers that were presented at the symposium (six US and six USSR) are reprinted in English in this volume in accord with the protocol signed by the delegation leaders on November 24, 1978.

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Land Treatment: Achieving Nutrient Removal by Recycling Resources

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Introduction

As it is throughout much of the world, land treatment is an established methodology for treatment and reuse of wastewaters in the United States. A discussion of land treatment for recycling loses meaning without a common understanding of background, terminology, and present concepts. Land treatment as we know it today represents a substantial advancement of technology over that of even a few decades ago. This advancement of technology includes many changes in terminology beginning with the term sewage farming and shifting to more specific terms describing alternative land treatment processes.

Background

A strong beginning for sewage farming in the United States was followed by a period of popularity and extensive use which continued into the first part of the twentieth century. This early concept of land treatment for recycling was based on farming with settled sewage as the source of nutrients and water. Use of raw sewage for farming was reasonably successful although there were odor problems and occasional transmission of disease to farm animals or humans. In the third decade of the twentieth century there was a definite shift away from sewage farming to inplant treatment of sewage with discharge of the partially treated wastewater directly to streams and lakes. This treatment and discharge approach to handling of sewage gained quick popularity. Most of the new sewage treatment systems built in the United States between 1930 and 1970 adopted the nonrecycle alternative of inplant treatment and discharge. Pollution of streams and lakes increased dramatically during this period. It quickly became apparent that discharge of partially treated sewage and the plant nutrients it contained into streams and lakes was unsatisfactory. By 1960 there was a strong movement to either provide more nutrient removal through inplant processes or to reconsider nutrient recycling through processes such as sewage farming. A concerted effort was initiated to advance the state-of-the-art for both of these alternatives. Others are covering several aspects of improving inplant treatment to remove nutrients. I will address nutrient recycling by land treatment. I use the term land treatment instead of sewage farming because the advances in technology have made it possible to select from several land treatment alternatives.

Terminology

These recent advances in technology have been accompanied by coinage of many terms to describe land treatment alternatives. The United States Environmental Protection Agency (US EPA) has adopted three terms from the many available. I shall describe each of these to avoid confusion while covering the nutrient recycling aspects of land treatment systems. The terms to be described are "slow rate," "rapid infiltration" and "overland flow."

Slow Rate: Slow rate land treatment is often referred to as irrigation. The term slow rate land treatment is used to focus attention on wastewater treatment rather than on irrigation of crops. However, in slow rate systems, vegetation is a critical component for managing water and nutrients. The applied wastewater is treated as it flows through the soil matrix, and a portion of the flow percolates to the groundwater. Surface runoff of the applied water is generally not allowed. When the primary objective of the slow rate process is treatment, the hydraulic loading is limited either by the infiltration capacity of the soil or the nitrogen removal capacity of the soil-vegetation complex. If the hydraulic capacity of the site is limited by a relatively impermeable subsurface layer or by a high groundwater table, underdrains can be installed to increase the allowable loading. Grasses are usually chosen for the vegetation because of their high nitrogen uptake capacities.

When the crop yields and economic returns from slow rate systems are emphasized, crops of higher values than grasses are usually selected. In the West, application rates are generally between 1 and 3 inches/wk (2.5 to 7.6 cm/wk), which reflect the consumptive use of crops. Consumptive use rates are those required to replace the water lost to evaporation, plant transpiration, and stored in plant tissue. In areas where water does not limit plant growth, the nitrogen and phosphorous in wastewater can be recycled in crops. These nutrients can increase yields of corn, grain sorghum, and similar crops while providing an economic return.

Slow rate treatment is generally capable of producing the best results of all the land treatment systems. The quality values shown in Table 1 can be expected for most well designed and well operated slow rate systems.

Rapid Infiltration: In rapid infiltration land treatment (frequently referred to as infiltration-percolation), all of the applied wastewater percolates through the soil, and the treated effluent eventually reaches the groundwater unless intercepted by drains or recovery wells. The wastewater is applied to rapidly permeable soils, such as sands and loamy sands, by spreading in basins or by sprinkling, and is treated as it travels through the soil matrix. Vegetation is not usually used, but there are some exceptions. Removals of wastewater constituents by the filtering and straining action of the soil are excellent. Suspended solids, BOD and fecal coliforms are almost completely removed in most cases as shown in Table 1. The nitrogen content of the reclaimed water is a nutrient source which can be recycled when the water is recovered through drains or wells for use as an alternate source of irrigation water.

Overland Flow: In overland flow land treatment, wastewater is applied over the upper reaches of sloped terraces and allowed to flow across the vegetated surface to runoff collection ditches. The wastewater is renovated by physical, chemical, and biological means as it flows in a thin film down the relatively impermeable slope. Biological oxidation, sedimentation, and grass filtration are the primary removal mechanisms for organics and suspended solids. Biological nitrification/denitrification is a major mechanism for achieving the nitrogen removal shown in Table 1. Overland flow is a relatively new treatment process for municipal wastewater in the United States. As of August 1976, only three relatively small, full-scale municipal systems have been constructed. These are located in Oklahoma, Mississippi, and South Carolina.

Present US EPA Concepts

A changing attitude toward conservation and recycling in the United States also includes emphasis on management of nutrients in wastewaters. The concept of recycling the nutrients in sewage is gaining in popularity as opposed to expending energy and using resources to provide more nutrient removal without use of the nutrients. The three land treatment options which were described in the previous section offer varying capabilities to recycle nutrients while reducing energy requirements for treatment. Slow rate systems which emphasize crop production provide the greatest opportunity for recycling nutrients. Tailoring the preapplication treatment to conserve the nutrients while eliminating pathogenic organisms is a key to effective design of these slow rate systems. Rapid infiltration systems provide for less recycling opportunity because the emphasis is on use of minimum land areas to reclaim the wastewater. The opportunity to recycle nitrogen comes if the reclaimed water is recovered and used as an alternate source of irrigation water. Overland flow provides for partial recycle of nutrients while emphasizing reclamation of wastewater with discharge to a surface water. Although the reclaimed water could be recovered for a secondary use such as irrigation, it would be a comparatively poor source of nutrients. The attractiveness of overland flow is its simplicity, low cost, and suitability for use on low permeability soils.

The US EPA sees this concept of three types of land treatment systems (slow rate, rapid infiltration, and overland flow) as complimentary technologies which furnish land treatment alternatives suited to most site conditions regardless of the dominant soils at the site. This ability to select alternatives provides flexibility for tailoring nutrient recycling and wastewater reclamation to site conditions and a variety of project purposes.

Recycling With Slow Rate Systems

Slow rate systems offer a direct pathway for recycling of the plant nutrients contained in wastewaters. Although there are several options for the design of a project, the basic premise is to use wastewater to provide the major nutrients and the supplemental water, if needed, to grow a crop. We use the term slow rate because the emphasis is on maximum recycling rather than providing supplemental water. Slow rate systems are usually designed with the amount of wastewater applied or the amount of nitrogen applied as the limiting factor. It is only recently that consideration of the amount of nitrogen applied became an important design factor. All of the older systems were designed on the basis of the amount of wastewater applied as the limiting factor.

Designing and operating slow rate systems on the basis of applied wastewater alone has been very successful for producing crop yields equalling or exceeding yields obtained by using commercial fertilizers and well or surface waters for irrigation. Such practices have provided economical wastewater treatment while recycling substantial quantities of plant nutrients. Many examples of operating slow rate systems in the United States are described in a recent US EPA survey of land treatment systems([1]). Some of these systems have been studied in detail over the last decade to determine long term results for crop production, changes in soil properties, and changes in groundwater quality under the site.

Systems at Bakersfield, California, and Lubbock, Texas, are typical examples of systems that have produced consistently high yields for a variety of crops for up to 50 years. Approximately 1,000 hectares is being irrigated at each of these locations. Recent investigations(2) at these sites have confirmed that consistently high yields are still being harvested after many decades of operation. There have been no adverse affects on the soil properties that would reduce crop yields or harm the quality of the crops. While crop yields are excellent and soil properties remain good, there has been considerable loss of nitrate nitrogen to the local groundwater. This has been a consequence of failing to consider the amount of nitrogen applied as an important and perhaps a limiting design factor.

A similar slow rate system at Roswell, New Mexico, has operated over a comparable period of time. The amount of wastewater applied at this site has been substantially less than that at Bakersfield, California, or at Lubbock, Texas. The crops have used virtually all of the nitrogen applied over the 30 plus years of continuous wastewater irrigation. Consequently, there has not been a comparable loss of nitrate nitrogen to the shallow groundwater under the irrigated fields.

Newer slow rate systems in the United States are being designed on the basis of nitrogen applied as well as the amount of wastewater being applied to the crops. A large system of about 2100 hectares at Muskegon, Michigan, is a good example of this approach to the design of a system. While still early in its operating life, the Muskegon system is providing valuable quantitative information on nutrient recycling(3). This system, designed to serve a population equivalent to 430,000 people, recycles water and fertilizer nutrients to grow corn. Operating at a population equivalent of 300,000 people in 1976 the 2,200 ha farm produced 144,000 hl (400,000 bu) of corn. Revenues from the sale of the crop offset much of the costs to operate all components of the wastewater management system. About 90 metric tons of phosphorus, 230 metric tons of nitrogen, and 250 metric tons of potassium were recycled by crop uptake or stored in the soil as the wastewater was reclaimed.

In addition to the obvious project benefits from the recovery and recycling of nutrients through crop production and crop sales to help offset the system's overall operating costs, there are important water quality benefits provided by this wastewater treatment system. Improvements have already been observed in the water quality of the lakes that had been receiving the wastewater discharges now passing through the land treatment system. This marriage of water quality improvement and wastewater treatment with nutrient recovery and recycling has been achieved without adverse effects on the groundwater underlying the project site or surface waters into which the renovated water is discharged.

These examples of nutrient recycling with slow rate systems offer a cross-section of operating systems in the United States. They depict the flexibility available for design and operation of slow rate systems and many new systems are being planned or constructed at the present time. The concept of multiple use projects is gaining popularity in water short areas of the country where some projects include recreational use as well as nutrient recycling for crop production. Forest irrigation is an approach which is also receiving much attention as a means for maintaining native vegetation. The future of nutrient recycling by slow rate systems is very optimistic in the United States at the present time.

Recycling by Rapid Infiltration

Rapid infiltration seldom offers a direct pathway for recycling of plant nutrients contained in wastewaters. The major purpose of rapid infiltration is to reclaim wastewater and, by so doing, condition the wastewater for other uses. Recycling of nutrients by using the reclaimed water as an irrigation source is one of the practical uses of the reclaimed wastewater. If there is no need to make direct use of the reclaimed wastewater it is usually released to surface waters. A system at Phoenix, Arizona, is a good example of using the reclaimed water for irrigation and a system at Lake George, New York, is a good example of releasing the reclaimed wastewater to surface waters.

Phoenix, Arizona, is located in the southwestern part of the United States where water is in short supply. There is a need to make direct use of wastewaters to extend the limited water supply. Soil and geologic conditions are favorable for using rapid infiltration to reclaim wastewater by providing advanced treatment of a conventional treatment plant effluent and storing it underground where it can be recovered as needed for crop irrigation. Research conducted at Phoenix, Arizona, shows the renovated water following rapid infiltration to contain less than 2 mg/l of suspended solids and BOD, 7.5 mg/l of total nitrogen, 0.2 mg/l of phosphate phosphorus, and 6 fecal coliforms per 100 ml. After the rapid infiltration process has removed the SS, BOD, and fecal coliforms the reclaimed water is suitable for irrigation of crops including those which may be eaten raw. This rapid infiltration approach is an economical process with a low energy demand because it relies on natural processes in the soil to reclaim wastewater for subsequent use for many uses including crop irrigation.

Lake George, New York, is located in the northeastern part of the United States where surface waters are plentiful. The need to reuse wastewaters is low yet it is necessary to eliminate pollution of surface waters by discharging conventional treatment plant effluents and the nutrients they contain into these surface waters. The rapid infiltration system at Lake George was designed to prevent discharge of wastewater (even after treatment) into clear and beautiful Lake George. The reclaimed water which discharges into the lake through springs after rapid infiltration is of very high quality(5). Suspended solids, biochemical oxygen demand, and fecal bacteria are completely removed by filtration through the soil. Phosphorus has been reduced to a concentration of less than 0.1 mg/l and the nitrogen concentration is about 7 mg/l. The reclaimed water discharge which averages 4.3 l/sec is so pure that it has no adverse affects on the quality of water in a small brook into which the reclaimed water is discharged by the springs. With no specific need to recycle the nutrients by direct reuse of the wastewater, the rapid infiltration system provides low cost and energy conserving use of natural processes to reclaim wastewater and prevent the pollution of a clean lake.

These examples of operating rapid infiltration systems under radically different climatic conditions and for substantially different objectives show the utility of this technique. In water short Phoenix, Arizona, the emphasis is on conservation of a water resource for subsequent reuse including the recycling of nutrients. In Lake George, New York, where high quality water is plentiful, the emphasis is on preventing the direct discharge of pollutants into the lake. The situation and site conditions at Lake George are not conducive for recycling nutrients while the situation and conditions at Phoenix are ideal for nutrient recycle and multiple use of reclaimed wastewater.

Nutrient Recycling by Overland Flow

The overland flow process provides a land treatment alternative suited to locations with low permeability soils. Such sites are unacceptable for rapid infiltration and unacceptable to marginally acceptable for slow rate systems. Overland flow systems must be vegetated with selected species of grasses. Nutrient recycling into a hay crop is an integral part of system operation although the fraction of the applied nutrients removed in the vegetation is a small fraction of that applied in the wastewater. Use of overland flow for treatment of domestic wastewaters is a little used but rapidly developing technology in the United States. The principal use in the United States has been for treatment of high strength food processing wastewaters as described in a research study of an operating system at Paris, Texas (6).

Research on treatment of domestic wastewaters by overland flow has been conducted at a US EPA laboratory in Ada, Oklahoma, since 1971. The results of these studies (7)(8) show that overland flow can provide simple and low cost treatment of raw sewage as well as serving as a process for providing advanced treatment following either primary or secondary treatment (9). As a process for treatment of raw sewage overland flow eliminates the generation and consequent handling of sludges while producing an excellent quality effluent. Using a land area of one hectare per 600 persons to treat raw wastewater the overland flow process produces an effluent containing less than 10 mg/l of SS and BOD, and about 5 mg/l of total nitrogen and total phosphorus. It is these aspects of the overland flow process for land treatment which make it very attractive as an energy saving and low cost alternative for reclaiming wastewater where treatment and discharge is of greater importance than direct recycling of nutrients. Removal of nitrogen is attributed to natural nitrification/denitrification. The dual step of nitrification/denitrification in a system open to the atmosphere is achieved by having a double layer effect in the water flowing as a sheet down the slope. Nitrogen balances show that 1150 kg/ha of nitrogen are applied yearly. Of this amount 100 kg are discharged in the system effluent and 150 kg are removed by harvesting the crop. This leaves about 900 kg/ha which is converted to gaseous nitrogen and released to the atmosphere. This represents a substantial loss of a resource which could be recycled as a plant nutrient but it is achieved by natural processes without an input of additional energy. The future is bright for use of overland flow as a wastewater reclamation alternative for year round operation in geographic regions with mild to warm climates or for seasonal operation in colder regions.

Summary

Nutrient removal by land application furnishes many options to the user. Slow rate systems offer the best opportunity for direct recycling of the nutrients for crop production as the nutrients are stripped from the wastewater. After preapplication treatment which reduces fecal organisms to desired levels while conserving plant nutrients the wastewater is used directly for irrigation of selected crops. Rapid infiltration offers the best opportunity to reclaim water while using the least amount of land area so opportunities for direct recycling are minimal. The opportunity for combining nutrient recycling with rapid infiltration comes through recovery of the reclaimed water for subsequent reuse as an irrigation water. The purpose of using rapid infiltration is to make the reclaimed wastewater suitable for irrigation of all types of crops as well as providing temporary underground storage. Overland flow offers a limited opportunity to recycle nutrients while providing a low cost and energy saving option for reclaiming wastewater. It is especially suited for low permeability soils which make rapid infiltration impossible and slow rate systems impracticable. This combination of complimentary approaches makes nutrient removal, or more appropriately nutrient management by land treatment, an attractive alternative to removal of nutrients by using additional inputs of chemicals or energy to conventional inplant treatment. The future is very promising for nutrient management and recycling by land treatment in the United States.

Table 1

Expected Quality of Treated Water From Land Treatment Processes mg/L

	Slow rate[a]		Infiltration[b]		Overland flow[c]	
Constituent	Average	Maximum	Average	Maximum	Average	Maximum
Biochemical Oxygen demand (BOD)	2	5	2	5	10	15
Suspended solids (SS)	1	5	2	5	10	20
Ammonia nitrogen as N	0.5	2	0.5	2	0.8	2
Total nitrogen as N	3	8	10	20	3	5
Total phosphorus as P	0.1	0.3	1	5	4	6

a. Percolation of primary or secondary effluent through 5 ft. (1.5m) of soil.

b. Percolation of primary or secondary effluent through 15 ft. (4.5m) of soil.

c. Runoff of comminuted municipal wastewater over about 150 ft. (45m) of slope.

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Works for Final Purification of Municipal Wastes

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The necessity of final purification of municipal wastes has arisen as a result of continuously increasing water consumption and population growth reaching a point where waste effluent, even after a complete biological treatment characterized by a decrease of the biologically permissible concentration and suspended matter to 10-15 mg/l and by the beginning of the process of nitrification, can no longer ensure that a required quality of the water is maintained at the place of water consumption. A long-term forecast shows that if complete biological treatment plants are constructed and effectively exploited in every town, the pollution of water reservoirs will increase. Therefore one of the directions of the development of research and techniques of waste treatment is the development of works for additional removal of pollutants from biologically treated municipal effluent.

The most pressing problem now is the general reduction of pollutants in discharged waste waters in terms of the content of suspended matter and biologically permissible concentration.

At present the most sophisticated method for their removal is by filtering through gauze devices and granular materials allowing water of different quality to be produced by using devices of different construction.

Investigations carried out using filters consisting of granular materials have shown that the suspended matter of biologically treated effluent is characterized by strong forces of cohesion with the filtering medium, i.e. by capacity for forming aggregates on the surface of grains in the medium and for accumulating in the initial layers of the medium. Because of this, filters used for final purification of biologically treated effluent are characterized by a curvilinear (parabolic) increase in the loss of head in contrast to a rectilinear one for filters in water treatment plants. These peculiarities of suspended matter lead to the necessity of filtering the water in the direction of the decreasing size of the filtering material and of using water-air washing. This can be achieved by employing double- or multi-layer filters and upflow gravel-sand filters.

Double-bed anthracite-sand filters are exploited at the Zelyonograd Aeration Plant of the City of Moscow. For a filtering rate of 5 m/hr there is a decrease in the suspended matter on the filters from 10-12.5 to 1.3-2.9 mg/l and that of biologically permissible concentration from 5.8-6.8 to 2 mg/l.

However, it is better, in our opinion, to use upflow gravel-sand filters for final effluent purification, since they have a greater mud capacity and the loss of head increases most slowly along a comparatively flat curve.

The utilization of the upflow filtration principle has allowed the filter mud capacity to be increased by a factor of up to 2 or 2.5 as compared with conventional fast filters and the filtering rate to be increased up to 12-13 m/hr under normal conditions and up to 14-15 m/hr under forced conditions. For water with a suspended matter content of up to 20 mg/l the duration of a filtration cycle was over 24 hr. With an increase in the content of the suspended matter to 30 or 40 mg/l the filters worked steadily, the duration of the filtration cycle remaining sufficiently large, up to 12 hours.

The latter circumstance allows one to reduce, when utilizing upflow filters, not only the area of the filters themselves but also the volumes of the aerator tanks. The fact is that in the Soviet Union it is customary to calculate the volumes of secondary precipitation tanks for the staying of sewage during two hours from a maximal hourly inflow. This restricts the amount of activated sludge in the aerator tanks to 1.8-2.5 g/l, with a permissible removal of the suspended matter from the secondary settling tanks reaching 10 to 15 mg/l (in the absence of works for final effluent purification). If upflow filters are used, it is possible to increase the permissible removal from the secondary tanks to 30 or 40 mg/l, which makes it possible to increase the amount of the activated sludge in the aerator tanks to 3 or 4 g/l, without changing the volumes of the secondary precipitation tanks, and to reduce the volumes of the aerator tanks.

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Upflow filters ensure a stable removal of impurities with a short-term increase in the content of the suspended matter to 60 or 80 mg/l, which insignificantly affects the quality of the filtrate with a duration of the filtration cycle of not less than eight hours.

A three-stage water-air washing is recommended. The volume of water required for one washing is 1.5-2 per cent of the volume of water to be treated.

Under the above working conditions upflow filters ensure a decrease in the content of the suspended matter of up to 80 or 90 per cent, of the biologically permissible concentration of up to 60 to 70 per cent and of the chemically permissible concentration of up to 25 or 45 per cent.

The application of 40 to 70-mu gauze microfilters for the final purification of effluent can ensure a decrease in the content of the suspended matter by 50 or 60 per cent and that of the biologically permissible concentration by 20 or 30 per cent with a filtering rate of 20 to 30 m/hr. Unlike granular material filters, there is no decrease in the dissolved oxygen.

It can be seen from the above data that microfilters are less effective in the removal of pollutants than filters with a granular material, especially in reducing the biologically permissible concentration. In view of their low cost, however, microfilters will find application in those places in the Soviet Union where a decrease in the suspended matter is of primary concern. It is also possible to use microfilters at water treatment plants with biological ponds as independent works or final purification works.

There are biological ponds for final effluent purification with both natural and artificial aeration in the Soviet Union. The advantages of biological ponds—thorough results of treatment for the biologically permissible concentration (up to 5 or 6 mg/l with a simultaneous decrease in the nitrogen and phosphorus compounds by 90 per cent), the simplicity of maintenance and relatively low cost of construction—lead to their application even at water treatment plants of major cities, such as Gorki and Kuibyshev, each with a population of more than a million. The lack of unoccupied areas and the severe climatic conditions in the greater part of the Soviet Union prevent them from being practiced on a larger scale. Among the disadvantages of biological ponds one should mention a high content of suspended matter (mainly phytoplankton) in the emerging effluent, which can be successfully retained by microfilters.

If there are surface-active materials (or detergent) in the effluent, one can use froth flotation for the final purification of the water from the surface-active materials and general pollution. The degree of final purification of the effluent by froth flotation depends on the kind of surface-active materials used, on their froth-forming capacity. It is of stable character in towns, however, 70 to 90 per cent of the surface-active materials and 50 to 60 per cent of the content of the suspended concentrations being removed. The intensity of barbotage is 35 $m^3/(m^2hr)$ for a process lasting 30 min. The collected froth is damped down and moved on to be treated together with the precipitate.

Froth flotation is the cheapest method, the effect of the treatment in terms of the common pollutants is intermediate with respect to those for filters with a granular material and microfilters, but its obvious advantage is a decrease in the surface-active materials. As to its disadvantage it is that froth flotation can be applied only if there are surface-active materials (not less than 2-3 mg/l) in the biologically treated water.

In recent years many scientists have been giving their attention to the removal of biogenous elements from biologically treated effluent when the latter is discharged into closed water reservoirs or reservoirs with slightly flowing water. For the Soviet Union this problem is also becoming an urgent one since it has more than 150 big reservoirs alone, the volume of each exceeding 100 mill. m^3. The total volume of the artificial reservoirs reaches 500 km^3, almost 10 per cent of the total annual runoff of the country or 22 per cent of the volume of all reservoirs in the world.

By the present time, reagent methods for the removal of phosphorus have been studied in the Soviet Union, such as simultaneous precipitation and reagent filtration. For this purpose one can use traditional mineral coagulants, viz. aluminum and iron salts, lime, etc. It has been established that the best results for the removal of phosphorus using simultaneous precipitation are obtained with 20 mg/l batches of coagulant (Fe_2O_3) the residual phosphate content having amounted to less than 0.5 mg/l. However, increasing the coagulant batch led to the oppression of the activated sludge which was manifested by a decrease in its dehydrogenation activity. It has been established that iron coagulant exerts toxic effect when introduced in amounts exceeding 5 mg per 1 g of the ashless material of the sludge, the effect becoming aggravated with sludge loads exceeding 500 mg of biologically permissible concentration per 1 g of ashless material.

In the case of simultaneous precipitation the sludge index was observed to decrease to 50 cm/g, which makes it possible to increase the sludge batch in the aerator tanks to 6 or 7 g/l. In the above process the growth of activated sludge increases by 25 or 35 per cent. When salts of divalent iron are used, the blast intensity should be increased by a factor of 1.4.

Upflow filters can be used to reduce the total amount of pollutants after the simultaneous precipitation, the technological parameters being the same as for non-reagent filtration.

When using filters it is possible to introduce reagents before the filters (reagent filtration). Then the coagulant batch is reduced to 5 or 8 mg/l (for Fe_2O_3) as compared with simultaneous precipitation. But even these amounts of coagulant increase the sludge load on the filter, which leads to the necessity of decreasing the filtering rate to 5 or 7 m/hr and the duration of a cycle of filtration to 8 or 12 hr. The effect of the treatment is 80 to 90 per cent for phosphorus, 70 to 90 per cent for the suspended matter, 70 to 90 per cent for the biologically permissible concentration and 35 to 50 per cent for the chemically permissible concentration.

The last method is the most expensive in terms of capital costs but attains the greatest effect in removing pollutants.

Investigations are in progress in the Soviet Union into the removal of nitrogen compounds from biologically treated effluent by physiochemical and biological methods, but they have not been completed yet

The high requirements for the quality of effluent treatment for water reservoirs make it economically advisable to use thoroughly treated municipal wastes in industrial water systems. At present, because of the lack of reliable data for a sanitary-hygienic estimation of thoroughly treated effluent, it appears advisable not to use it in food, meat and milk or pharmaceutical industries.

In most water-consuming industries (power engineering, non-ferrous and ferrous metallurgy, oil industry, etc.) the greatest amount of water is needed for use as heat-carrier, for which purpose municipal effluent can be used to advantage after proper treatment with due regard for precautions excluding any contact of the operating staff with the water.

The current lack of scientifically based requirements for the quality of water used in various industries restrains the utilization of treated municipal effluent as industrial water.

OPERATION OF 5 MGD MODEL AT THE ADVANCED WASTEWATER TREATMENT IN PISCATAWAY, MARYLAND

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Objective

The project objectives were to operate a 5 mgd tertiary wastewater treatment plant to demonstrate high efficiency removal of phosphates, organic carbon and suspended solids. The process train included lime clarification, filtration and granular carbon absorption.

Abstract

A 5 mgd tertiary wastewater treatment plant was constructed to demonstrate treatment of effluent from a 5 mgd step aeration activated sludge plant. The two-stage high lime process with intermediate recarbonation, filtration and activated carbon adsorption operated continuously at the design rate for 36 days. A single-stage low lime process with filtration and activated carbon adsorption operated for 89 days. The combined secondary and tertiary treatment removed 97% of BOD, TSS and P in the raw waste-water. Capital cost of the 5 mgd tertiary treatment system was 4.7 million dollars and operating costs were estimated as 36 cents per 100 gallons of wastewater.

Detailed Description of Secondary Treatment Facility

The primary and secondary treatment facilities at Piscataway consist of two parallel systems (capacity = 5 and 25 mgd) with a common solids handling system. Raw wastewater is presently pumped to the plant from four pumping stations with the total capacity of 75 mgd. Although the total feed system has the capacity to feed 75 mgd, neither the sewer taps nor the plant have the capacity to collect or treat this volume of wastewater.

A schematic diagram of the liquid treatment facilities is shown in Figure 1. Although the entire secondary plant is presented, only those unit processes associated with the 5 mgd system feeding the Model Plant will be discussed. The 5 mgd secondary system as operated during the grant is shown in Figure 2.

The raw sewage is manually split and fed to the three grit chambers. The effluent from the single grit chamber for the 5 mgd system passes through two 3/8" barminutors equipped with automatic rotating cutters. Four centrifugal pumps, each rated at 1750 gpm, are piped to the open channel following the barminutors to provide a constant flow of 5 mgd through the 5 mgd secondary section into the Model Plant. Since the three chambers are interconnected, all flow in excess of 5 mgd is diverted to the two grit chambers in the 25 mgd system.

The degritted effluent is split at the primary inlet well and fed to two parallel primary settling tanks. The primary solids are wasted by gravity to the primary solids collection well. At the collection well, the primary solids are combined with the scum from the surface skimmers and flow by gravity to the sludge thickener.

Primary effluents from the two clarifiers are combined and flow by gravity to the step aeration basins. Settled solids from the secondary clarifiers are recycled to the

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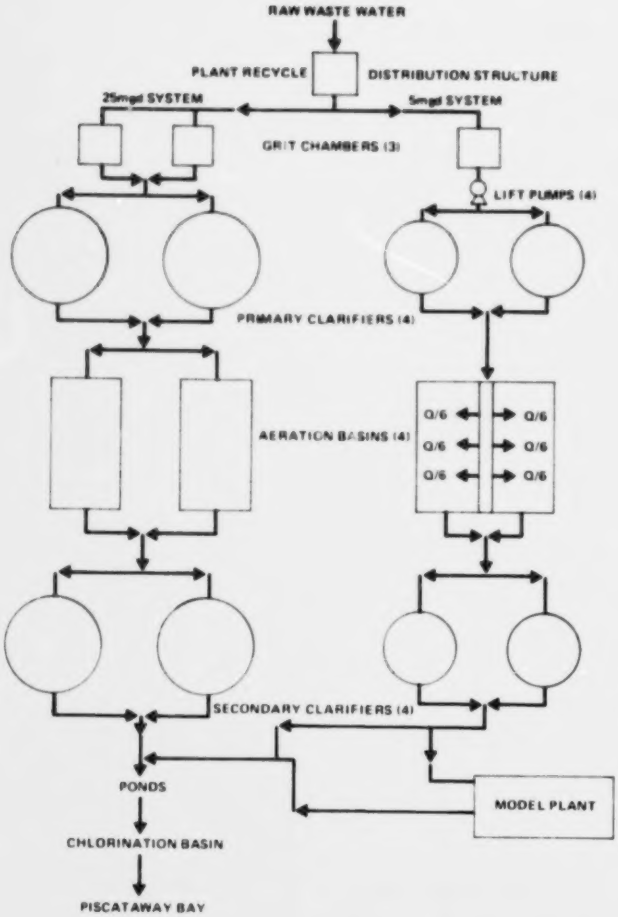

RAW WASTE WATER
PLANT RECYCLE
DISTRIBUTION STRUCTURE
25mgd SYSTEM
5mgd SYSTEM
GRIT CHAMBERS (3)
LIFT PUMPS (4)
PRIMARY CLARIFIERS (4)
Q/6
Q/6
Q/6
Q/6
Q/6
Q/6
AERATION BASINS (4)
SECONDARY CLARIFIERS (4)
PONDS
MODEL PLANT
CHLORINATION BASIN
PISCATAWAY BAY

Figure 1. Flow schematic of the Piscataway secondary plant.

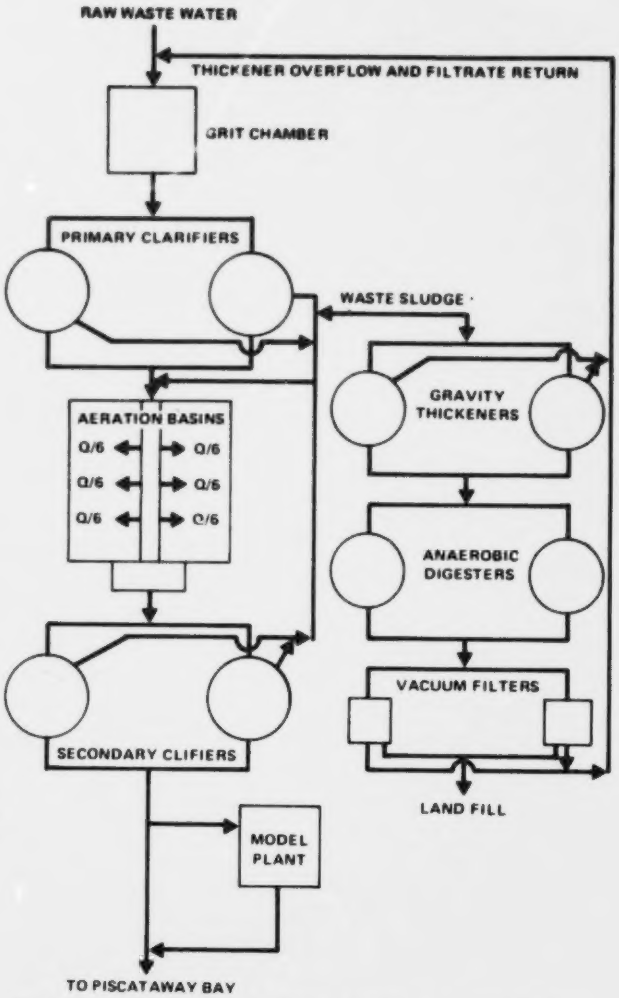

RAW WASTE WATER
THICKENER OVERFLOW AND FILTRATE RETURN
GRIT CHAMBER
PRIMARY CLARIFIERS
WASTE SLUDGE
AERATION BASINS
Q/6
Q/6
Q/6
Q/6
Q/6
Q/6
GRAVITY THICKENERS
ANAEROBIC DIGESTERS
VACUUM FILTERS
SECONDARY CLIFIERS
LAND FILL
MODEL PLANT
TO PISCATAWAY BAY

Figure 2. Schematic of the 5 mgd system of the Piscataway plant secondary plant.

head of the two reactors to maintain an average MLSS concentration of 2000 mg/l. Normally, the recycle flow was maintained at 33% of total with 4.5 hours detention. Normal operation called for control of the DO at approximately 2.5 mg/l O_2.

The mixed liquor flows by gravity to the secondary settler inlet structure where the flow is split and fed to two settlers. At a flow of 2.5 mgd/settler, the units provide 2.75 hours of detention time at a surface loading of 650 gpd/sq. ft. Centrifugal pumps return the settled solids to the reactors. Excess flow not reporting to the Model Plant is diverted through a Parshall flume to the polishing ponds which were installed at the plant as an interim upgrading step pending completion of the 25 mgd system.

The waste solids from both the primary and secondary settlers flow by gravity to the thickeners' inlet structure from which the flow is directed to four gravity thickeners; two basins 35 ft in diameter and two basins 55 ft in diameter.

The underflow from the thickeners was pumped to two anaerobic digesters. The capacity of the digesters was the controlling factor in the solids handling system. Overflow from the thickeners are recycled to the head of the plant. Because of the limited capacity of the digester this overflow was high in suspended solids which effected the operation of the primary settlers.

Following anaerobic digestion the under flow is vacuum filtered. The cake is trucked to a farm for spreading. Because of health requirements, only stabilized sludge can be vacuum filtered and disposed of on the land.

Tertiary Treatment

Operation of the tertiary treatment plant consisted of treating the secondary effluent from the 5 mgd step aeration system by lime clarification, dual-media filtration and granular activated carbon adsorption. The solids from the chemical clarification system were gravity thickened and dewatered by solid bowl centrifuges. The dewatered cake was recalcined in a multiple hearth furnace. The exhausted granular activated carbon was thermally regenerated in another multiple hearth furnace.

Lime Treatment

In lime clarification of relatively low alkalinity wastewaters, two options are available: two-stage high lime with intermediate recarbonation or single-stage low lime. Since the alkalinity of the Piscataway wastewater is less than 150 mg/l as $CaCO_3$ and adequate flexibility was incorporated into the design of the plant, both the single and two stage systems were evaluated.

In the two-stage high system shown in Figure 3, lime slurry, wastewater and recycled settled solids are mixed in the rapid mix zone of the first clarifier to reach pH 11.5. At pH 11.5 nearly all of the magnesium is precipitated as magnesium bycloride which serves as an excellent coagulent aid. Settled solids are recycled to increase the rate of precipitation and size of the precipitated solids. As a result of the high lime dose, excess calcium ions are present in the effluent from the first clarifier. Carbon dioxide is added in the two stage recarbonation unit to reduce the pH to approximately 9.5. Carbon dioxide was supplied by the recalcination of calcium carbonate which produces calcium oxide (CaO) and carbon dioxide (CO_2). This allows the excess calcium ions to be precipitated as calcium carbonate. These solids are captured in the second clarifier with the aid of a small dose (18/l/for FeCl3 of ferric chloride).

In this single stage lime system, shown in Figure 4, the pH of the wastewater is increased to 10.5 by the addition of lime in the rapid mix zone of the first clarifier. Most of the phosphorus is precipitated along with the available carbonate. However, without the precipitation of magnesium hydroxide, good clarification does not occur and a coagulant aid is required. Ferric chloride is added for this purpose to the rapid mix zone of the single clarifier. The concentration of excess calcium ions in the effluent from the single-stage is approximately 50 to 60 mg/l as Ca and recovery by the addition of carbon dioxide is not economically feasible.

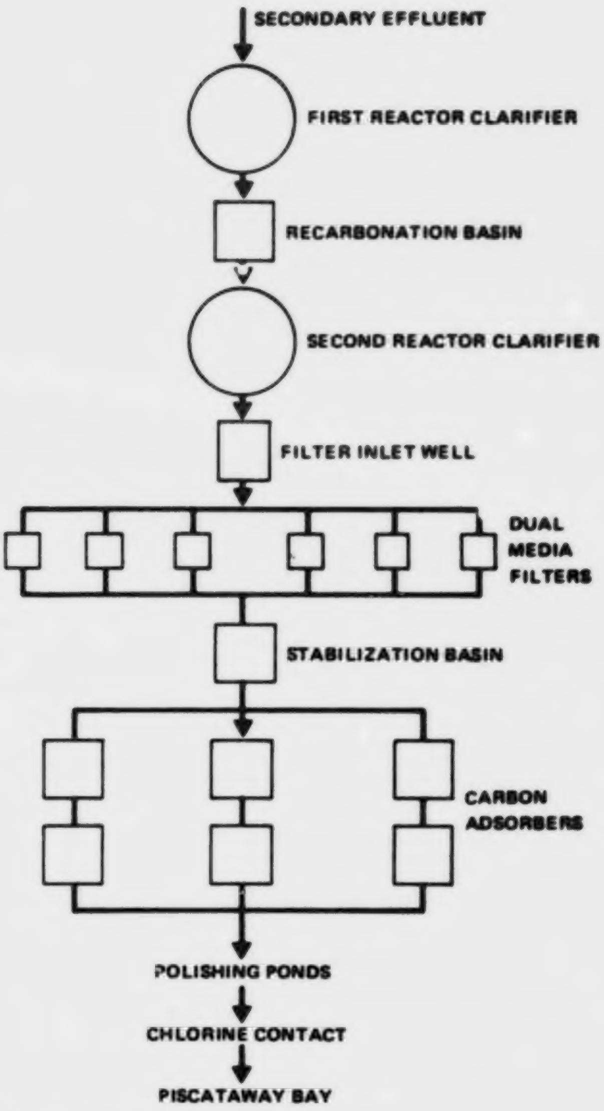


Figure 3. Two-stage high lime tertiary process.

Dual Media Filter

In the two stage lime system effluent from the second clarifier (ph 9.0) was reduced to pH 8 in the filter inlet well and fed directly to the dual media filter. Care must be taken to combat the pH since excessive calcium ions will produce cementation of the filter media as calcium carbonate. In the single storage system, the recombustion basin is used to reduce the pH to 8 thus converting the insoluble calcium carbonate to the soluble ion forms.

The effluents were fed to the dual media gravity filter for removal of particulate materials including phosphorus and organics. It was felt that reduction of the pH below 8 would result in insolubilization of the phosphorus thus allowing it to pass through the filter. The filter media consists of 18 inches anthrocite coal (effective size 0.85 to 0.95 mm) over 6 inches of fine sand (effective size 0.40 to 0.45 mm). The backwash water from both the filter and carbon calcium is stored and returned to the head of the plant at a controlled rate.

The filter effluent is reduced to pH 7.5 in the water stabilization tank by the addition of carbon dioxide. The reduction in pH is necessary to meet stern discharge standards and optimize activated carbon absorption.

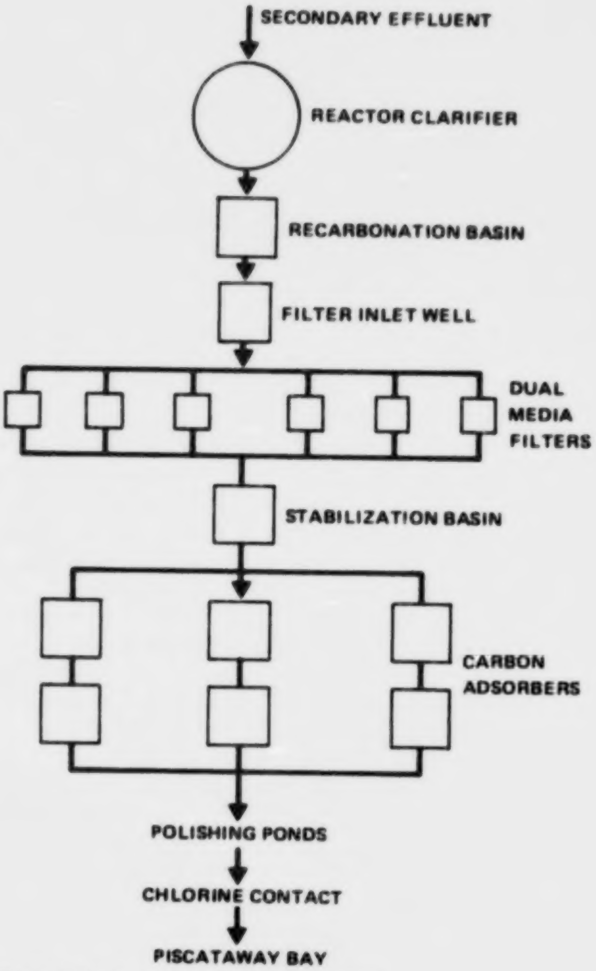

SECONDARY EFFLUENT
REACTOR CLARIFIER
RECARBONATION BASIN
FILTER INLET WELL
DUAL MEDIA FILTERS
STABILIZATION BASIN
CARBON ADSORBERS
POLISHING PONDS
CHLORINE CONTACT
PISCATAWAY BAY

Figure 4. Single stage low lime tertiary process.

Activated Carbon Adsorption

The final unit process in the tertiary treatment scheme is activated carbon adsorption. The carbon adsorption system consists of six downflow pressurized vessels arranged in sets of two to provide three parallel trains of two columns each. A depth of 16 ft of activated carbon provides 18 minutes of Empty Bed Contact Time (EVCT) per column. The effluent from the stabilization tank is pumped at a rate of 6.5 gpm/sq ft through a packed granular activated carbon bed which absorbs soluble organic materials from the wastewater. The adsorbed organics serve as a food source for bacteria which multiply on the carbon to produce biological slimes. The biological activity, if controlled, can substantially increase the life of the activated carbon. To control the activity and to prevent excessive pressure losses through the packed carbon beds, backwash and surface wash of the carbon columns are necessary. The backwash rate is 15 gpm/sq ft. The effluents from the three carbon systems are collected in a common line and flow to the polishing ponds. The effluent from the ponds receiving carbon effluents is combined with the effluents from the ponds containing Piscataway secondary effluent, chlorinated and discharged to Piscataway Bay.

Carbon Regeneratio

Following exhaustion of the activated carbon, the carbon is removed from the column for thermal regeneration. The carbon regeneration system is shown in Fig. 5. The exhausted carbon is removed from the activated carbon column through the four funnels located in the underdrain of each column (Fig. 6) and hydraulically carried to the exhausted carbon storage tank. It is transferred, at a controlled rate, into a multiple hearth furnace where the regeneration takes place in 4 stages:

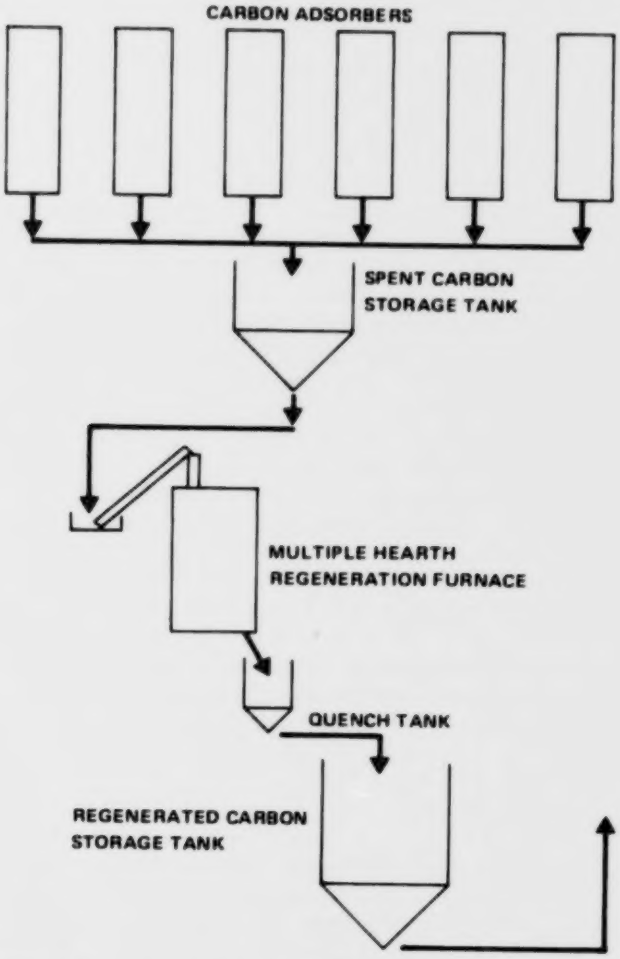
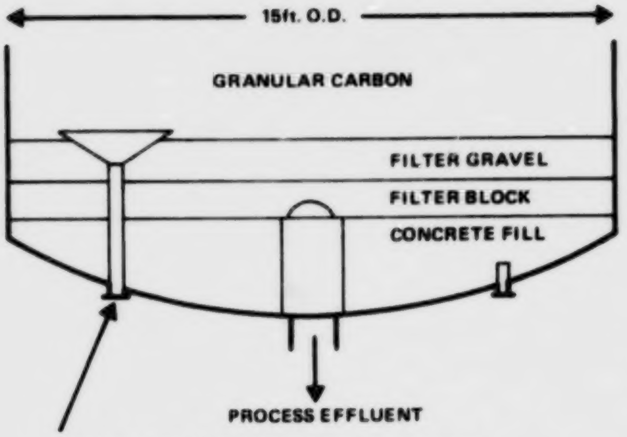

CARBON ADSORBERS
SPENT CARBON STORAGE TANK
MULTIPLE HEARTH REGENERATION FURNACE
QUENCH TANK
REGENERATED CARBON STORAGE TANK

Figure 5. Flow schematic for carbon regeneration.



4- Carbon draw-off hoppers used to transfer carbon from the adsorbers to the spent storage tank

Figure 6. Cross section of carbon adsorber underdrain.

1. The wet carbon is dried by simple evaporation at temperatures above 200°F.

2. Upon application of heat to the carbon grains at temperature above 600°F, the high molecular weight impurities on the carbon will crack to produce gaseous hydrocarbons, hydrogen and water vapor.

3. The final regeneration step is the gasification of the residue from the pores of the carbon grains. This is accomplished using steam (approximately 1 lb steam/lb dry carbon) at temperatures between 1700 and 1850°F. The gaseous products of the reactions are carbon monoxide and hydrogen.

4. The regenerated carbon is finally cooled rapidly to ambient temperatures by water sprays.

Lime Handling and Recovery

The chemical solids from the lime clarification system must be either recovered for reuse or subjected to ultimate disposal. Laboratory tests and material balance calculations show that at Piscataway in the single-stage system, the solids production is approximately 2 lb/1000 gal with a calcium carbonate concentration of 50%. The two-stage system will produce approximately 4 lb/1000 gal with a 75% calcium carbonate concentration. One of the objectives of this study was to determine the cost of solids handling and calcination both with and without recovery of the calcium carbonate.

Results of Two-Stage High Lime Evaluation

The operation of the two-stage high lime system was sustained at the design flow rate for 36 days in October and November, 1973.

Secondary Operation

The operation of the step aeration activated sludge system described in Table 1 corresponds to the 36 days of operation of the tertiary facility and was typical of that achieved during the two years of the grant period. The mixed liquor suspended solids concentration of 2133 mg/l is an average of samples taken at the quarter points along the reactor. As seen by the average decrease in alkalinity from 135 to 97 mgl/l as CaCo$_3$3, the activated sludge system was nitrifying. The reduction in alkalinity is a result of the production of nitric acid during nitrification. The variations in alkalinity and TKN in the secondary effluent is shown in Figure 7. The SRT of 6.6 days is based on the average wasting rate for the 36 operating period. The average concentrations of BOD and suspended solids for the raw wastewater, primary effluent and secondary effluent are presented in Tables 2 and 3. The effect of the recycle of solids from the soluble handling system to the grit chamber is reflected in the high values for these parameters in the primary effluent.

Table 1

Operating Conditions of the Piscataway Secondary Plant During the High Lime Process Evaluation

Daily Flow, mgd	5.675
Detention Time, hr	4.4
MLSS, mg/l	2133
Recycle Rate, %	37
Solids in Recycle, mg/l	7970
Waste Rate, 1000 gal/day	38.1
SVI, ml/gm	96
SRT, days	6.6
F/M, lb BOD$_5$/lb MLVSS	0.41
Raw Wastewater pH	7.2
Secondary Effluent pH	7.4
Raw Wastewater Temperature, °F	65
Raw Wastewater Alkalinity, mg/l CaCo$_3$	135
Secondary Effluent Alkalinity, mg/l CaCO$_3$	97

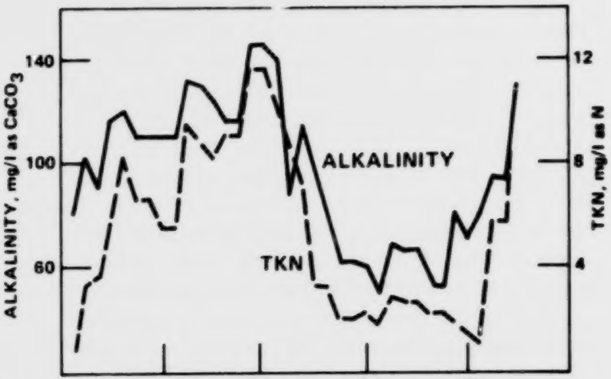


Figure 7 Comparison of Alkalinity and TKN of Secondary Effluent.

Table 2

Removal of Biochemical Oxygen Demand (BOD 5 Day) During the High Lime Process Evaluation

	mg/l	% Removal
Raw	141.0	—
Primary	145.0	—
Secondary	16.5	88.3
Lime clarified	5.9	95.8
Filtered	5.7	96.0
Carbon Adsorption	4.0	97.2

Note: Secondary Plant recycle enters between the raw sample point and the primary clarifier.

Table 3

Removal of Suspended Solids During Evaluation of the High Lime Process

	mg/l	% Removal
Raw	121	—
Primary	183	—
Secondary	27.5	77.3
Lime Clarified	21	82.6
Filtered	6	95.0
Carbon Adsorption	2.5	97.9

Note: Secondary Plant recycle enters between the raw sample point and the primary clarifier.

Table 4

Removal of Total Phosphorus (AS P) During Evaluation of the High Lime Process

	mg/l	% Removal
Raw	7.90	—
Primary	9.60	—
Secondary	3.50	55.7
Lime Clarified	0.26	96.7
Filtered	0.20	97.5
Carbon Adsorption	0.10	98.7

Note: Secondary Plant recycle enters between the raw sample point and the primary clarifier.

The phosphorus concentrations are given in Table 4. During the evaluation period for the tertiary lime treatment systems, 80 mg/l alum was being added to the aeration tanks of the 25 mgd activated sludge system to improve removals of phosphorus and suspended solids. Return streams to the head of the plant containing high concentrations of alum sludge and precipitated phosphate entered the 5 mgd secondary system and produced high phosphorus concentrations in the primary effluent. The overall effect of the return stream was in increase in phosphorus removal through the old 5 mgd secondary plant. During the 36 days of operation, the secondary plant removed 55.7% of the incoming phosphorus. The average nitrogen concentrations are given in Table 5, but do not reflect the variability as seen in Fig. 7.

Table 5

Removal of Nitfogen* Compounds During Evaluation of the High Lime Process

	NH_3	TKN	NO_3	No_2	Total N
Raw	13.1	16.2	1.1	0.1	17.4
Primary	—	—	—	—	—
Secondary	4.5	5.2	6.7	.4	12.3
Lime Clarified	5.4	6.0	6.4	.3	12.7
Filtered	4.1	—	5.9	—	—
Carbon Adsorption	2.9	3.3	8.3	.1	11.5

*All values mg/l as N

Tertiary Treatment

During the 36 days of continuous operation the influent flow to the Model Tertiary Plant averaged 4.589 mgd. The hydraulic loadings to the unit processes based on influent flow plus recycle are presented in Table 6. The recycled plant water was 11.3% of the total flow. A summary of the plant recycle flows is presented in Table 11. The chemical usage during the operation of the system is presented in Table 8. As seen, the calcined lime accounted for 75% of the lime added and 67% of the available calcium oxide. Carbon dioxide was supplied by the recalcination furnace 22 of the 36 days. Liquid carbon dioxide was supplemented for a portion of 14 days. Three methods were used to determine the material balances around the clarification and solids handling system using the following data:

1. Chemical analyses of the liquid streams and measured daily influent flow rates.

2. Chemical analyses of the sludge streams and measured sludge flows.

Table 6

Loading Rates During Evaluation of the High Lime Process

Secondary Plant		
Flow 5.7 mgd		
Primary Clarifiers	2-60 ft dia.	1008 gpd/sq ft
Secondary Clarifiers	2-70	741 gpd/sq ft
Model AWT Plant		
External Flow and Recycle =	Total Flow	
4.586 mgd + 0.588 mgd =	5.174 mgd	
First Stage Reactor Clarifier	1-80 ft dia	1029 gpd/sq ft
Second Stage Reactor Clarifier	1-70 ft dia.	1344 gpd/sq ft
Dual Media Filtration (5 units)	242 sq ft/unit	4276 gpd/sq ft or 2.97 gpm/sq ft
Carbon Adsorption		
Column Set #1 Avg. Flow 2.063 mgd		8.1 gpm/sq ft
Column Set #2 Avg. Flow 1.420 mgd		5.6 gpm/sq ft
Column Set #3 Avg. Flow 1.309 mgd		5.1 gpm/sq ft

All loadings are based on an average flow of 5.174 mgo.

Table 7

Plant Recycle Flows During High Lime Evaluation

Sources	Total Gallons/Day	% of Flow
1. Filter Backwash		
38,720 gal × 6 filters/day	232,320	4.49
2. Carbon Column Backwash		
24,017 gal × 4 columns/day	96,068	1.86
3. Recalcination Furnace		
164 gal/min × 1440 min/day	236,160	4.56
4. Misc. — (centrate, pump sealing water, flushing & wash water)	23,452	0.45
Total	588,000	11.36

All percentages are based on an average flow of 5.174 mgd.

Table 8

Chemical Usage in the High Lime Process

Lime	
Total Virgin Pounds	137,079
Average Available Lime Index (ALI.), %	87
Available CaO, lb	119,259
Average Daily Usage, lb	3,138
Average Daily Dose, mg/l	87
Total Recalcined Pounds	426,618
Average Available Lime Index (ALI.), %	60
Available CaO, lb	255,971
Average Daily Usage, lb	6,731
Average Daily Dose, mg/l	170
Ferric Chloride ($FeCl_3$)	
Total Pounds Added to Clarifier, lb	23,496
Average Daily Usage, lb	618
Average Daily Dose, mg/l	17.8
Polymer Usage (Centrifuge Only)	
Total Pounds Used	568.3
Average Daily Usage, lb	15.0
Pounds Polymer/Ton of Dry Sludge	0.25

3. Total solids concentrations and measured sludge flows.

The dual media filters were not effective in removing additional materials with the exception of suspended solids. The calculated removal of suspended solids in the filters was 15 mg/l. However, since the suspended solids in the influent to the filter system consists mainly of precipitated calcium carbonate, a portion of which is solubilized in the filter inlet well, an accurate efficiency of the filters alone cannot be determined. However, if one assumes that 15 mg/l of suspended solids is captured in one 24-hour run at 1 mgd, an estimate of the efficiency of the filters can be determined. A total of 125 lbs of solids would be captured in each filter for a loading of 0.52 lb/sq ft/cycle. By using a termination headloss of 8 ft, then the loading can be expressed as 0.065 lb/sq ft/ft of headloss. These loadings are quite reasonable considering that the units contain 24 inches of filter media compared to 36 inches of media in most designs.

Cost Analysis

The major purpose of this project as conceived by EPA and WSSC was to gather reliable data with regard to operating costs and operational problems. Accomplished on a small pilot scale and the purpose of the Piscataway Model Plant project was to build and evaluate a larger system. The costs for the Model Plant were high for the following reasons:

1. Data are based on the startup period where equipment was being modified and adjusted.

2. Operators were unfamiliar with the plant. The majority of the staff had little training in the wastewater field or prior experience in treating wastewater.

3. Processes were not operated within optimum ranges of efficiency.

Capital Expenditures

The engineer estimated the cost of the 5 MGD model plant at 3.2 million dollars. As shown in Table 10, the final cost of the Model Plant was 4.68 million dollars. Table 11 shows a more detailed breakdown of the costs in which the EPA's final share was 3.1 million dollars, the State of Maryland paid 0.3 million and WSSC paid 1.1 million dollars. Table 12 includes the cost of engineering services. Included in Table 13, 14 and 15, unit process and equipment costs have been detailed. Note in Table 13, that an attempt was made to determine capital cost of the low line system. Also in Table 13, note that the engineering services have been included as complete plant costs and are not broken down for the individual processes.

Table 9

Removals of Chemicals Oxygen Demand (COD) and Total Organic Carbon (TOC) During Evaluation of the High Lime Process

	COD mg/l	TOC mg/l as C
Raw	—	—
Primary	—	—
Secondary	34.8	12.3
Clarified	—	13.4
Filtered	24.7	7.5
Carbon Adsorption	13.4	1.8

Table 10

Capital Costs of the Model Plant

General Contractor (Main Contract)	$3,037,100
Change Orders	210,338
Furnace Contractor	529,000
Activated Carbon	148,356
Centrifuges	66,480
Electric Substation	153,800
Sub Total	$4,145,074
Engineering Services	535,243
Total	$4,680,317

Table 11

Distribution of Capital Costs of the Model Plant

U.S. Environmental Protection Agency	
Under Project 17080DZY	
Research and Development	EPA's share 75%
$3,200,000 × .75	= $2,400,000
Under Project WPC - MD - 233	
Additional Facilities	$1,320,453
55% EPA's share	
$1,320,453 × .55	= $ 726,249
25% State of Md. share	
$1,320,453 × .25	= $ 330,113
Washington Suburban Sanitary Commission	
Main Contract	$ 800,000
Additional Facilities	$ 264,090
	$1,064,090

Table 12

Costs of Engineering Services for the Model Plant

Preliminary Engineering Design and Report	$ 28,449
Study of Advanced Treatment	7,281
Preparation of Plans and Specifications	288,511
Construction Services	190,095
Preparation of Operations Manual	20,907
Total	$535,243

Table 13

Capital Costs for the Model Plant Unit Processes

Process	High Lime Cost	Low Lime Cost*
Lime Clarification	$1,340,190	$ 991,000
Filtration & Carbon Adsorption	1,150,354	1,150,354
Solids Handling	1,099,247	291,930
Carbon Regeneration	401,483	401,483
Sub Total	$3,991,274	$2,834,767
Electrical Substation	153,800	** 109,198
Engineering Services	535,243	** 380,022
Total	$4,680,317	$3,323,987

*Low lime cost is based on a calculated estimate.
**Both of these numbers are proportional to the sub totals.

Table 14

Breakdown on Capital Costs for the Model Plant Unit Processes

Clarification		
	Excavation	$ 52,700
	Concrete Work	187,400
	Mechanical Work	237,000
	Electrical Work	28,000
	Painting	10,000
	Other Work	20,300
		$ 535,400
Filtration		
	Steel Shelves	$ 118,000
	Underdrains	35,000
	Filter Media	8,000
	Mechanical Work	29,500
	Piping and Valves	32,000
	Painting	3,000
	Electrical Work	600
	Other	7,100
		$ 232,200
Carbon Adsorbers		
	Steel Tanks and Supports	$ 232,000
	Filter Bottom	28,000
	Mechanical Work	26,000
	Piping and Valves	75,000
	Electrical Work	6,000
	Painting	7,900
	Other Work	1,800
		$ 367,700
Operations Building		$1,076,900
Centrifuge Area & Cake Handling System (excluding centrifuge)		33,800
Lime Handling System		100,200
$FeCl_3$ and Polymer System		43,200
Motor Control Center and Power Distribution		105,500
Instrumentation		308,800
Miscellaneous Equipment, Steel and Clean-up		226,337
		$1,0

Table 15

Capital Cost Breakdown for the Model Plant Equipment

General Contract	Cost	Date of Completion
Reactor Clarifier	$ 294,000	October 72
Recarbonation Basin & Associated Tanks	92,400	October 72
Lime Pumping Stations	81,000	October 72
Thickener & Pumping Station	68,000	October 72
Operations Building	1,076,900	April 72
Dual Media Filters	194,000	April 72
Backwash Tanks	78,4000	April 72
Stabilization Tank	50,500	April 72
Activated Carbon Adsorbers (excluding carbon)	247,500	April 72
Carbon Storage Tanks	30,500	April 72
Lime Bins	31,500	May 72
Lime Slaker System	25,700	February 73
Assorted Lime Handling Systems	43,000	February 73
Lime Cake Handling Systems	33,800	October 72
Miscellaneous Plant Equipment	61,600	April 72
Chemical Feed System (Ferric & Polymer)	42,200	October 72
Steam Generator	13,500	April 72
Platform & Structures	69,200	April 72
Motor Control Center & Power Distribution	105,500	March 72
Instrumentation	308,800	February 73
Sitework & Piping	88,100	April 72
Sub Total	$3,037,100	
Change Orders	210,338	February 73
Furnace Contracts	529,000	March 73
Activated Carbon	148,356	October 72
Centrifuges	66,480	July 72
Total	$3,991,274	

Conclusions

This report covers results from the operation of a 5 mgd tertiary wastewater treatment plant used to upgrade the quality of effluent from a conventional secondary plant using step aeration activated sludge. The tertiary plant used either two-stage high lime or single-stage low lime followed by dual-media filtration and granular activated carbon adsorption. Major conclusions from the demonstration project are as follows:

1. The high lime process using an average dosage of 257 mg/l CaO and 18 mg/l $FeCl_3$ significantly reduced residuals of BOD, TSS and P in the secondary effluent. BOD was reduced from 16.5 mg/l to 4.0 mg/l; TSS was reduced from 27.5 mg/l to 2.5 mg/l and P was reduced from 3.50 mg/l to 0.10 mg/l.

2. The low lime process using an average dosage of 113 mg/l CaO and 25 mg/l $FeCl_3$ produced removals of BOD, TSS and P comparable to those obtained with the high lime process.

3. Tertiary treatment did not significantly affect total N residuals in the secondary effluent.

4. Carbon losses from regeneration of three columns under less than optimum conditions were estimated as 8-10%.

4. Operating costs for tertiary treatment in this demonstration plant were in the range of 29-36 cents per 1000 gallons. These are considered to be unusually high because data are based on the startup period when the plant was not at optimum efficiency and operators were not familiar with the plant.

6. A highly competent, well trained, technical staff is required to successfully operate a complex tertiary wastewater treatment plant. Extra efforts must be made to select, train and retain personnel.

Aerated Filters for Tertiary Treatment of Secondary Effluents

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Aerotanks of various modifications are the most popular units for removal of organics from municipal and industrial waste waters where their concentration is expressed as biochemical oxygen demand (BOD) value.

Theoretically aerotanks are able to reduce BOD value down to 2-3 mg/l, but in our country from the economical point of view it is recommended to treat effluents down to BOD value 15 mg/l, and in some countries the standard degree of effluents treatment in aerotanks is even lower.

When biologically treated effluents are being discharged into streams of low rate of flow in comparison with this discharge, or into water bodies used for fish propagation, etc., the degree of treatment on aerotanks does not satisfy rules of discharge, and water inspectorate authorities demand in designs provision for a tertiary treatment installation.

For some years economically developed regions are experiencing shortage of service water for industrial needs. The water shortage can be alleviated by usage of treated municipal effluents for some industrial processes, for example for equipment and apparatus cooling, mountain waste and cinder hydraulic transportation, gas cleaning, etc. In such cases tertiary treatment of effluents is also necessary.

Thus the necessity of tertiary treatment of secondary effluents is caused not only by hygienic demands to improve quality of the effluents, but also by the rationality of usage of treated effluents for industrial water supply.

In connection with this in our country and elsewhere for some years investigations are being conducted with aim to develop new methods and designs of units for tertiary treatment of effluents. As concentration of suspended solids in biologically treated effluents is low - - usually within 15 - 50 mg/l — they can be removed by filters with an inert medium, such as quartz sand, granite rubble, etc. Such filters have been very popular for potable water treatment for a long time. But biologically treated effluents contain not only suspended pollutants but also colloidal and dissolved organics which are more dangerous from the sanitary point of view. Their concentration is also expressed by BOD value. Also they contain biogenic elements, in particular nitrogen and phosphorus. Such pollutants practically are not removable by treatment of effluents on conventional filters with a granular medium. Moreover, dissolved oxygen of aerobically treated effluents is completely absorbed by such filters. As dissolved oxygen concentration in secondary sedimentation tank effluents usually is not higher than 2 - 3 mg/l, the oxygen is absorbed by an upper layer of the filtering medium. Therefore in this layer, conditions appropriate for aerobic microorganisms and for biological processes are being created and these cause a partial reduction of BOD value. As the lower layers of the medium are deprived of oxygen, their anaerobic conditions are substituted for aerobic ones, captured suspended matter starts to decay, and by the end of filtration cycle the effluents acquire unpleasant putrescent odour and contain no dissolved oxygen. Effluents treated in conventional filters have to be saturated with oxygen before discharge into receiving waters.

At the Kharkov Department of VNIIVODGEO we have designed a granular filter for tertiary treatment of biologically treated effluents. Here filtration process is designed with due regard for peculiarities of such effluents and thus it obviates to a considerable extent shortcomings inherent to conventional filters.

It is known that the process of tertiary treatment of effluents in granular filters depends not only on physical-chemical and hydraulic factors, but also on concurrent biochemical factors as the bioflocculation phenomenon considerably influences the mechanisms of filtration. Some types of microorganisms are barely tolerant to turbulence occurring in aerotanks, but they can form a biological film on surfaces of solid bodies and this film very efficiently absorbs colloid and dissolved organics from effluents, provided the dissolved oxygen concentration is sufficient for their biological development.

Investigations of physical fundamentals of filtration processes of a low-concentration suspension of activated

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sludge, i.e. effluents of secondary sedimentation tanks after aerotanks, give justification to the idea that the granular filter supplied with air or oxygen would be suitable for biochemical processes throughout the depth of filtering medium, and thereby it would be possible to ensure tertiary removal not only of suspended solids but also of dissolved and colloidal organics. Concentrations of these substances are expressed as BOD value.

It is recommended to conduct tertiary treatment of effluents in two stages: preliminary sedimentation of activated sludge particles in the unaerated layer of the filter followed by filtering through the aerated layer.

We have developed for this purpose units of various designs. One of them (Fig. 1) is a two-tier filter. It consists of the tank 1 separated into two tiers (lower 2 and upper 4) by the perforated plate 3. Effluents of a secondary sedimentation tank are delivered by the pipeline 8 into the lower unaerated layer of the filtering medium where suspended particles are retained. The effluents from the lower tier are passing through the perforated plate of a special design 3 into the filtering medium of the upper tier 4 supplied with air or oxygen through the pipeline 6 when the filter is in operation. The treated water is collected and discharged by channels 5. The filtering medium of both tiers can be washed either jointly or separately. At separate washing the wash water of the lower tier is discharged through the pipeline 7.

When filtering, introduction of oxygen and nutrients in the form of residual organics remaining in the secondary effluents induces formation of a biological film on granular particles of the medium. The film consists of microorganisms intensively adsorbing and mineralizing pollutants. The tertiary treatment in a short time is ensured by a large area of the interphase contact (about 50 times larger than in rubble filters), by intensive forms of microorganisms, and by aerobic conditions.

Experimental investigations have been conducted in laboratory and pilot scale on effluents of the Kharkov aeration plant with aim to establish design, construction, and technology parameters. The effluents delivered for tertiary treatment contained 15-25 mg/l suspended solids, 15-20 mg/l BOD, 90-120 mg/l COD, 2-3 mg/l DO. The filters were charged with quartz sand of 1.0-1.8 mm grains diameter, depth of the charge layers were 1000 mm for each layer. The hydraulic load amounted from 5 to 13 m^3 per m^2/hr.

Treatment and analysis of experimental data have demonstrated that at filtration rate 6-7 m/hr and duration of filtering cycles 24-30 hrs, content of suspended solids was reduced by 85% in the lower tier of the filter. At the same rate of filtering through the aerated tier of the filter and at rates of air supply for upward filtering 1 m^3 per m^2/hr and for downward filtering 2.5 m^3 per m^2/hr, total BOD was reduced by 75-80%. At the same time the treated water was saturated with oxygen up to 7-8 mg/l.

Hydrobiological analyses have demonstrated that microorganisms populating the biofilm on the grains were active throughout the whole depth of the aerated medium.

Investigations of kinetics of BOD_{total} values reduction by different layers of the medium of the lower and upper tiers of the filter have demonstrated that introduction of air into the medium ensured aerobic conditions for biochemical processes and caused additional consumption of oxygen and reduction of BOD_{total} from 15-20 mg/l down to 4-5 mg/l. In unaerated layers of the filter biochemical consumption of oxygen, due to its shortage, was observed only in first layers of the medium when looking in direction of filtering (Fig. 2).

In the process of tertiary treatment in aerated filters the acid-base reaction of water was shifted by 0.3 units to the alkaline zone (in unaerated filters this shift amounted to 0.1 unit). This also indicates to biochemical activity of the microorganisms.

For investigation of influence of various factors on effluent treatment efficiency and for obtaining comparative characteristics of operation of aerated filters, pilot scale experiments were conducted on filters of various designs: two-tier aerated and unaerated filters, conventional grit filters with upward and downward delivery of effluents, reaerated filters, aerated filters with the microfilter.

Factors of operation of the units are given in the table. It is readily seen that the aerated granular filters provided the highest reduction of concentrations of the main kinds of pollutants.

On the basis of these investigations the main construction, design and technological parameters have been determined: filtering rate 7 m/hr, charge of the quartz sand of 1-1.8 mm coarseness; depth of medium layers 1000 mm for each layer; aeration intensity in downward filtering 2.5 m^3 per m^2/hr, in upward filtering 1 m^3 per m^2/hr; filtering cycle duration 24-28 hrs; washing intensity 16-17 l per s/m^2 at 6-8 min. duration; supporting layers, distribution, collection and discharge of water have to be designed in accordance with the construction standards and guidelines CHHII II-31-74.

When conducting experiments, concentrations of dissolved oxygen in the water were also determined. Water samples were taken from various points throughout the filter depth. The results of oxygen regime investigations are given on the diagram (Fig. 3). The diagram indicates that filtering through the lower unaerated layer of the medium reduced the dissolved oxygen concentration from 2-3 mg/l to 0.8-1.2 mg/l. In the upper aerated layer the treated effluents were saturated with oxygen up to 7-8 mg/l, thus the necessity of construction of special units for water aeration was eliminated.

Application of aerated filters for tertiary treatment of secondary effluents instead of conventional filters or biolagoons and aerating channels at a station of 25000 m^3 per day throughput would give an annual economic effect of 74 thousand roubles. But the main result consists in ensuring a steady high degree of treatment of effluents which can be discharged into a receiving water body of any type, or can be used for industrial water supply.

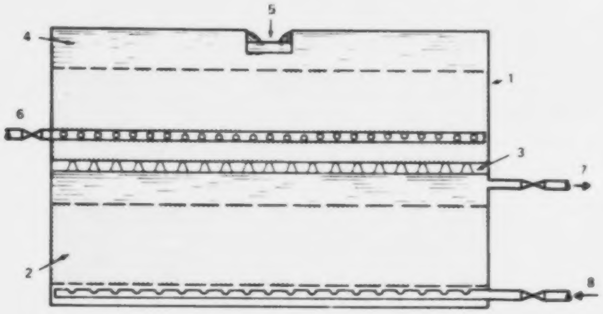

Fig. 1. Diagram of the two-tier aerated filter
1. tank, 2. lower tier, 3. perforated plate, 4. upper tier,
5. collecting channel for filtrate and washing water, 6. air inflow,
7. pipeline for filtrate and washing water, 8. inflow of effluents and washing water.

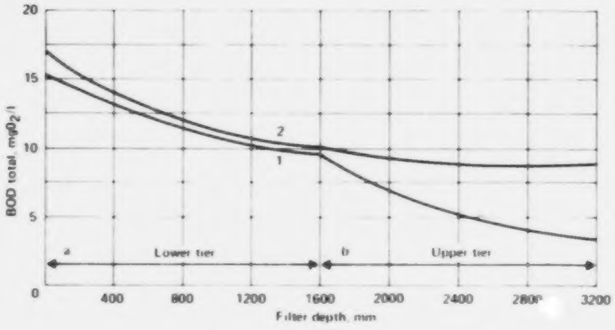

Fig. 2. BOD$_{total}$ value variation through the filter depth at filtering rate 7 m/hr
1. two-tier aerated filter
2. two-tier unaerated filter
a. lower tier, b. upper tier

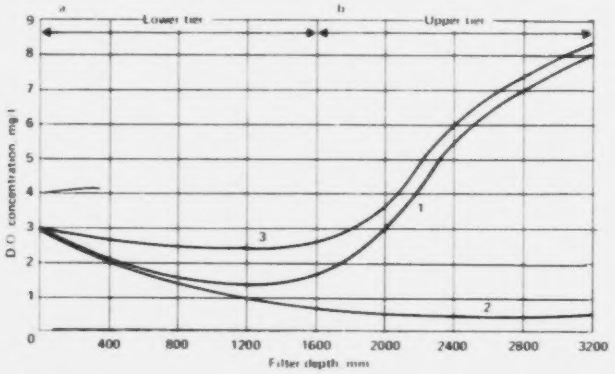

Fig. 3. Variation of DO concentration through the filter depth at filtering rate 7 m/hr
1. two-tier aerated filter
2. two-tier unaerated filter
3. aerated filter with the microfilter
a. lower tier, b. upper tier

Granular Carbon and Other Tertiary Treatment Processes

by

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Introduction

Previous papers (1,2,3) have discussed the need for and benefits of wastewater reclamation. The quality of many domestic water supplies throughout the world is inferior in comparison to many effluents from selected advanced wastewater treatment plants. It was discussed that, in the United States, although reclaimed sewage is not allowed for use as water supply, 145 of 155 cities having populations above 25,000 were found to have 17% to 18.5% sewage constituents by volume in their potable water supplies (4). At the 1977 Moscow symposium (1) various examples of advanced wastewater treatment facilities and their flowsheets were mentioned (see Table I).

Increased attention on toxic elements contained in water supplies has raised questions about the public health risks associated with the domestic water supplies of the U.S. According to Harris (5), communities deriving their water supplies from the southern portions of the Mississippi river had carcinogens in their potable water supplies. Accordingly, the question of the effectiveness of some systems for removal of trace toxic substances, including carcinogens, from wastewater prior to reuse needs careful consideration.

This paper will review two unit processes: carbon adsorption and lime treatment, and how their respective methods remove various elements posing health concerns to water reuse.

Health Considerations

Although economics may be attractive for water reuse, many would argue that countless examples of non-intentional water reuse in various communities are insufficient criteria to justify commitments to water reclamation plants. Problems such as health risks and esthetic considerations are crucial determinants for planning water reclamation facilities.

Many examples illustrate the existence of communities having water supply qualities that are less than ideal. For instance, a U.S. survey from 3,500 U.S. communities revealed that the water supplies consumed by 41% of these municipal systems had potable water of a poorer quality than the effluent from advanced wastewater treatment (6). Projected nationally, this is equal to 50 million people in the U.S. drinking tap water of a quality lower than the effluent from an advanced wastewater treatment plant. These communities do not appear to have any acute public health problems associated with consumption of drinking water that fails to meet the United Nations World Health Organization, WHO drinking water standards. WHO standards are shown in Table II. However, recent U.S. research indicates a health risk associated with drinking water containing low level residuals of trihalomethanes resulting from chlorination of natural and organic residuals. Where trihalomethane levels indicate synthetic organic chemical levels exceeding the recommended levels, granular activated carbon filtering of drinking water is required (7). Reclaimed water for non-potable uses such as cooling water make-up and industrial processes often allow even less stringent treatment levels than WHO standards.

The agents of public health concern found in wastewater can be divided into two categories: biological (infectious) and chemical compounds.

Infectious agents could include entities of bacterial, viral, or parasitic origins (8). The most important pathogens, including members of the genera Salmonella and Shigella, cause frequent outbreaks, but are controllable by adequate secondary treatment methods and are easily detected by standardized monitoring techniques. Viruses are not accurately monitored and present a more serious

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concern. The viral components of major health concern include:

1. enteroviruses (polio, coxsackie, Echo);
2. adenoviruses;
3. reoviruses; and
4. the agent of infectious hepatitus.

The second category of health-affecting agents includes the organic and inorganic chemicals. Many organic chemicals are those identified as problematic factors causing increased cancer risks. Table III is a list of organic chemicals identified by Cooper (8) in U.S. drinking water. Inorganic chemicals include the metallic ions. Table IV is adapted from Cooper (8) and is a list of toxic ions and their recommended concentration limits for safe drinking water as derived from available toxicology data and established dose tolerance levels.

The following discussion will analyze both carbon adsorption and lime treatment in respect to their individual and collective removal capabilities for these defined health hazards. Combinations of both unit processes have been shown to have the capability to remove most, if not all, of these agents from wastewater effluents.

Table 1

Examples of Advanced Wastewater Treatment Technology

Windhoek South West Africa	**Biological-algae-physical-chemical**	**WHO Standard**	**Tap water**
Lake Tahoe California	**Biological-physical-chemical**	**WHO Standard**	**Trout lake, swimming, irrigation**
Colorado Springs Colorado	**Biological-physical-chemical**	**BOD 11 ppm* SS 1.5 ppm**	**Industrial cooling water**
Rye Meads U.K.	**Extended-biological**	**BOD 10-5 ppm SS 10-5 ppm**	**Up to 20% of London tap water**
Central Contra Costa Sewage Treatment Plant	**Lime plus biological**	**BOD 2.0 ppm SS 1.0 ppm**	**Industrial cooling and processing**

*** parts per million**

Table 2

World Organization Requirements

World Health Organization Drinking Water Standards

Chemical Substances that Serve as Criteria for	**Maximum Acceptable Concentration**	**Maximum Allowable Concentration**
Potability		
Total solids	500.0 mg/l	1,500.0 mg/l
Color	5 units*	50 units*
Turbidity	5 units†	25 units†
Taste	Unobjectionable	
Odor	Unobjectionable	
Iron (Fe)	0.3 mg/l	1.0 mg/l
Manganese (Mn)	0.1 mg/l	0.5 mg/l
Copper (Cu)	1.0 mg/l	1.5 mg/l
Zinc (Zn)	5.0 mg/l	15.0 mg/l
Calcium (Ca)	75.0 mg/l	200.0 mg/l
Magnesium (Mg)	50.0 mg/l	150.0 mg/l
Sulphate (SO_4)	200.0 mg/l	400.0 mg/l
Chloride (Cl)	200.0 mg/l	600.0 mg/l
pH range	7.0-8.5	less than 6.5 or greater than 9.2
Magnesium + Sodium Sulphate	500.0 mg/l	1,000.0 mg/l
Phenolic substances (as phenol)	0.001 mg/l	0.002 mg/l
Carbon chloroform extract (CCE: organic pollutants)	0.02 mg/l	0.5 mg/l‡
Alkyl benzyl sulfonates (ABS: surfactants)	0.5 mg/l	1.0 mg/l
Health Hazards		
Nitrate as NO_3		45.0 mg/l
Fluoride		1.5 mg/l
Toxicity		
Phenolic substances		
Arsenic		0.05 mg/l
Cadmium		0.01 mg/l
Chromium (Cr hexavalent)		0.05 mg/l
Cyanide		0.2 mg/l
Lead		0.05 mg/l
Selenium		0.01 mg/l

*Platinum-cobalt scale.
†Turbidity units.
‡Concentrations greater than 0.2 mg/l indicate the necessity for further analysis to determine the causative agent.

Table 3

A Selected List of Potentially Toxic Chemical Agents Found in Drinking Water and Recommended Limit Concentrations

Chemical Agent	**Recommended Maximum Standard mg/l**
Arsenic	0.05
Barium	1.0
Cadmium	1.01
Chromium	0.05
Cyanide	0.20
Lead	0.05
Mercury	0.002
Nitrate	10.0*
Nitrite	1.0*
Selenium	0.01

*mg Nitrate or Nitrite-Nitrogen
Source: (8)

Trace Metals

Since many trace metals form insoluble hydroxides at pH 11, lime treatment eliminates these metallic constituents through coagulation or precipitation. Table V summarizes the lime removal efficiencies as condensed from the available literature (9). Filtration methods used to remove the residuals from lime coagulation will result in additional incremental removal of heavy metals. A full-scale example of an advanced water reclamation plant using lime treatment without carbon adsorption processes is the 113,500 m^3/day (30 million gallon/day)

Table 4

Organic Compounds Identified from Finished Drinking Water

Acetone
Acetophenone
Acetylene Dichloride
Benzene
Benzo Thiazole
Bromo Benzene
Bromo Chlorobenzene
Bromo Dichloromethane
Bromoform
Bromo Phenyl Phenyl Ether
Butyl Benzene
Camphanol
Camphor
Caprolactam
Carbon Tetrachloride
Chloro Benzene
Chloro Dibromo Methane
Chloro Ethoxy Ether
Chloro Ethyl Ether
Chloroform
Chloro Hydroxy Benzophenone
Bis-Chloroisopropyl Ether
Chloromethyl Ether
Chloronitro Benzene
Chloropyridine
Chloromethylethyl Ether
Dibromo Benzene
Dichloroethane
Dichloroethyl Ether
Dimethoxy Benzene
Hexachloro Benzene
Hexachloro Ethane
Hydroxy Adiponitrile
Isoborneol
Isocyanic Acid
Isopropanyl Isopropyl Benzene
Isopropyl Benzene
p-menth-l-en-8-pl
o-methoxy Phenol
2-Methoxy Biphenyl
Methyl Benzothiazole
Methyl Biphenyl
Methyl Chloride
Nitroanisole
Nitrobenzene
Octane
Pentane
Propylbenzene
Tetrachloroethylene
Toluene
Trichloroethane
Triglycodichloride
Thiomethylbenzothiazole
Vinyl Benzene
Dimethyl Naphthalene
Dimethyl Sulfoxide
Dinitrotoluene
Ethyl Benzene
Ethylene Dichloride
Exo-2-Camphanol

Source: (8)

Central Contra Costa Sanitation District (CCCSD) Water Reclamation Plant in Concord, California. The plant flowsheet, as shown in Figure 1, is designed for a possible expansion to handle up to 445,000 m^3/day (120 million gallon/day) (10, 1, 11). The influent characteristics are expected to average 220 mg/l BOD, 240 mg/l suspended solids, 30 mg/l nitrogen N, 11 mg/l total phosphorus as P. After treatment, the effluent values will be 3.5 mg/l, 6.0 mg/l, 2.0 mg/l and 0.5 mg/l, respectively (10). The effluent will be used by the following five industries: Phillips Petroleum, Shell Oil, Stauffer Chemical, Monsanto and the Pacific Gas and Electric power station. Although no data are available for heavy metal removal, it is expected to equal or exceed those high removal values as shown in Table IV.

A study at the University of Colorado (12) demonstrated that four trace elements (cadmium, chromium, selenium and silver) having particularly low allowable levels by USPHS drinking water standards were removed by lime coagulation from secondary effluent by 97% for the silver and 94% for cadmium ions. Using a combination of lime for coagulation and settling, sand filtration, activated carbon adsorption and an anion-cation exchange sequence demonstrated that (see Tables VI and VII) extremely high removal efficiencies (more than 98.6%) were achieved by the combination of these processes for selenium, chromium, silver, and cadmium ions. With only sand filtration and carbon adsorption, chromium, cadmium and silver were removed by at least 98.5% efficiency, with selenium having a 44.7% removal efficiency. This clearly suggests that using lime plus carbon adsorption will effectively remove toxic metals. The use of sand filtration and lime will help keep the carbon column from overloading with organics and trace metals for a longer time period before regeneration is necessary.

Table 5

Removal of Heavy Metals by Lime Coagulation and Recarbonation

Metal	Concentration Before Treatment mg/l	Concentration After Treatment mg/l	Final pH	% Removal
Antimony[a]			11	90
Arsenic[a]			11	<10
	23	23	9.5	0
Barium[a]		~1.3 (sol)[b]	11	
Bismuth[a]		.0002 (sol)	11	
Cadmium	Trace		11	~ 50
	0.0137	0.00075	>11	94.5
Chromium (+6)	0.056	0.050	>11	11
Chromium (+3)	7,400	2.7	8.7	99.9^+
	15	0.4	9.5	97
Copper	15,700	0.79	8.7	99.9^+
	7	1	8	86
	7	.05	9.5	93
	302	Trace	9.1	99^+
	15	0.6	9.5	97
Gold[a]		<.001 (sol)	11	90^+
Iron	13	2.4	9.1	82
	17	0.1	10.8	99^+
	2.0	1.2[c]	10.5	40
Lead[a]		<.001 (sol)[b]	11	90^+
	15	0.5	9.5	97
Manganese	2.3	<0.1	10.8	96
	2.0	1.1[c]	10.5	45
	21.0	0.05	9.5	95
Mercury[a]		Oxide soluble		<10
Molybden&m	Trace		8.2	~10
	11	9	9.5	18
Nickel	160	0.08	8.7	99.9^+
	5	0.5	8.0	90
	5	0.5	9.5	90
	100	1.5	10.0	99
	16	1.4	9.5	91
Selenium	0.0123	0.0103	>11	16.2
Silver	0.0546	0.0164	>11	97
Tellurium[a,d]		(<0.001?)	11	(?90+)
Titanium[a,d]		(<0.001?)	11	(?90+)
Uranium[e]		?		?
Zinc		.007 (sol)	11	90^+
	17	0.3	9.5	98

[a] The potential removal of these metals was estimated from solubility data.
[b] Barium and lead reductions and solubilities are based upon the carbonate.
[c] These data were from experiments using iron and manganese in the organic form.
[d] Titanium and tellurium solubility and stability data made the potential reduction estimates unsure.
[e] Uranium forms complexes with carbonate ion. Quantitative data were unavailable to allow determination of this effect.

South Lake Tahoe, which we have previously discussed numerous times, is a facility where physical-chemical treatment was added to conventional or secondary treatment. This water has been used to form Indian Creek Reservoir, a trout lake, for swimming and for grazing areas. This biological plus physical-chemical advanced wastewater treatment flowsheet is shown in Figure 1. This facility, which benefits from using a combination of lime treatment and carbon adsorption, produces an effluent that easily meets both United States Public Health Service and World Health Organization (WHO) drinking water standards (9). Table VII lists data of this effluent showing the concentrations of heavy metals.

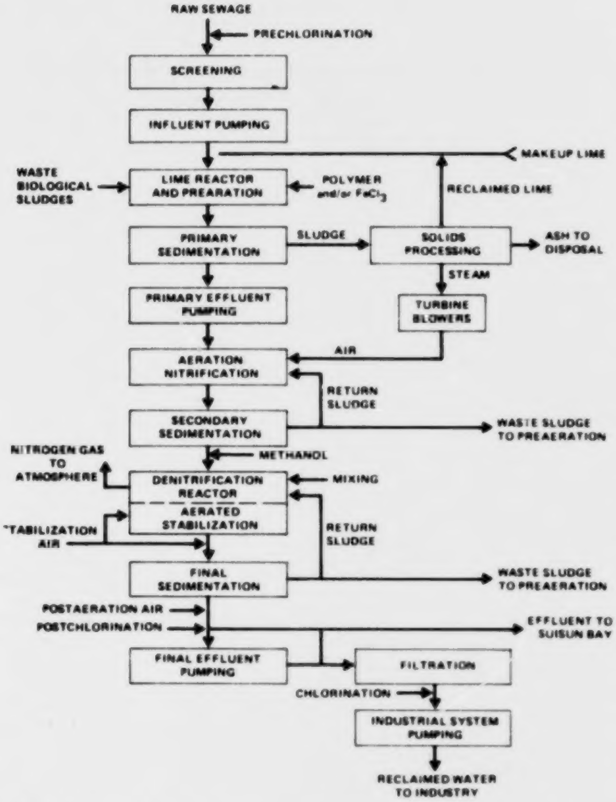
RAW SEWAGE
PRECHLORINATION
SCREENING
INFLUENT PUMPING
MAKEUP LIME
WASTE BIOLOGICAL SLUDGES
LIME REACTOR AND PREARATION
POLYMER and/or $FeCl_3$
RECLAIMED LIME
PRIMARY SEDIMENTATION
SLUDGE
SOLIDS PROCESSING
ASH TO DISPOSAL
STEAM
PRIMARY EFFLUENT PUMPING
TURBINE BLOWERS
AIR
AERATION NITRIFICATION
RETURN SLUDGE
SECONDARY SEDIMENTATION
WASTE SLUDGE TO PREAERATION
NITROGEN GAS TO ATMOSPHERE
METHANOL
DENITRIFICATION REACTOR
MIXING
AERATED STABILIZATION
'TABILIZATION AIR
RETURN SLUDGE
FINAL SEDIMENTATION
WASTE SLUDGE TO PREAERATION
POSTAERATION AIR
POSTCHLORINATION
EFFLUENT TO SUISUN BAY
FINAL EFFLUENT PUMPING
FILTRATION
CHLORINATION
INDUSTRIAL SYSTEM PUMPING
RECLAIMED WATER TO INDUSTRY

Figure 1. Central Contra Costa Sewage Liquid Process Flow Sheet Treatment Plant, Concord, CA.

Table 6

Removals of Trace Metals in Lime Coagulation-Settling Process

Trace Metal	Removal (%)
$Ag(Ag^{+})$	97.0
$Cd(Cd^{+2})$	94.5
$Cr(Cr_2O_7^{-2})$	9.3
$Se(SeO_3^{-2})$	16.2

Source (12)

Table 7

Removals of Trace Elements in Sand Filtration, Activated Carbon, Cation Exchange, and Anion Exchange

Trace Metal	Cumulative Removal After Given Process (%)			
	Sand Filtration	Activated Carbon	Cation Exchange	Anion Exchange
$Ag(Ag^{+})$	11.6	97.1	98.8	99.4
$Cd(Cd^{+2})$	6.2	98.8	98.5	99.1
$Cr(Cr_2O_7^{-2})$	2.5	96.6	98.5	98.6
$Se(SeO_3^{-2})$	9.5	43.2	44.7	99.9

Source (12)

Although the Tahoe facility was designed and built during a low energy cost era, it could be retrofitted and made less energy intensive through methods previously reported in this forum (1, 11). The energy recovery methods to which we are referring are improved dewatering, heat recovery, and pyrolysis, or co-disposal.

The latter option appears particularly attractive as, according to California and Nevada state law, all refuse solid waste must be transported out of the Tahoe Valley. Thus, the transportation savings would further augment the inherent energy recovery economics of pyrolysis.

Organic Compounds

Most of those organic constituents identified in Table II are adsorbed on carbon. Generally, the chemical nature of carbonaceous compounds is of relatively minor significance for the adsorption of these compounds by the activated carbon medium, as activated carbon has a preference for adsorbing organic compounds. Carbon filtration is particularly effective in removing trace organic substances associated with chronic health problems, in addition to eliminating odor and taste problems (9). Essentially, carbon adsorption in combination with other processes including pre- or post-chlorination and ozone treatment including pre-ozonation of carbon filter media and ozonation of water (13, 14) will eliminate organic residues. The success of the Tahoe process is qualitatively demonstrated at Tahoe where a rainbow trout fishery has been maintained for more than nine years using this effluent at Indian Creek Reservoir. Since trout fingerlings are extremely sensitive to chloro-organic compounds, one has a high degree of confidence that trace organics are effectively removed.

The effectiveness of carbon adsorption in removing trace organics is recognized by the U.S. Environmental Protection Agency (EPA) drinking water standards, which require carbon adsorption to insure the absence of these organic compounds which include many identified carcinogens (7). Since the cost of activated carbon can be quite expensive, carbon regeneration and reuse are most attractive. Numerous installations of carbon regeneration furnaces exist for carbon used in wastewater treatment, industrial processing and for potable water supplies. Table VIII is a list of the carbon regeneration facilities of one equipment manufacturer. The process utilizes a multiple hearth furnace (MHF) which raises the activated carbon to 1500°F., driving off the adsorbed carbon constituents into a gaseous phase. Water is used to quench the carbon prior to its being returned to the carbon filter. A loss rate of 5%-7% per regeneration cycle can be maintained. Cost estimates for virgin activated carbon were reported at approximately $.30 per pound (.46R/kg) while using this same price index would render the cost of regenerating activated carbon by MHF methods at $.04 per pound (.06R/kg) (9). As previously discussed, where levels of trace synthetic organic chemicals exceed baseline levels, activated granular carbon is required to remove these constituents (7).

Table 8A

Municipal Carbon Regeneration Furnace List

1963	6'0" O.D. × 6H	South Lake Tahoe, Ca.
1969	4'0" O.D. × 6H	Colorado Springs, Co.
1972	6'0" O.D. × 8H	Rocky River, Ohio
1974	7'9" O.D. × 5H	Derry Township, Pa
1974	12'9" O.D. × 6H	Vallejo, Calif.
1975	5'6" O.D. × 6H	Palo Alto, Santa Clara, Ca.
1976	16'9" O.D. × 6H	Tahoe-Truckee, Calif.
1976	12'9" O.D. × 6H	North Tonawanda, N.Y.
1976	(3) 10'9" O.D. × 6H	Alexandria, Va.
1977	10'9" O.D. × 5H	Nassau Co., N.Y.

Source: Envirotech Corporation

Table 8B

Industrial Carbon Regeneration Furnace List

User	Date of Installation	Size	Application
Spreckles Sugar, Salinas	1961	8'-6 × 8H	Beet Sugar Refining
Accent international, San Jose	1962	10' × 8H	Monosodium Glutamate
Spreckles Sugar, Mendota	1962	8'-6 × 8H	Beet Sugar Refining
Spreckles Sugar, Woodland	1962	8'-6 × 8H	Beet Sugar Refining
Holly Sugar Co., Calif.	1963	8'-6 × 8H	Beet Sugar Refining
Everglades Sugar, Florida	1964	8'-6 × 6H	Bone Char
Florida Sugar, Florida	1964	6' × 6H	Cane Sugar Refining
Supreme Sugar, Louisiana	1964	9'-3 × 6H	Cane Sugar Refining
Spreckles Sugar, Arizona	1965	8'-6 × 8H	Beet Sugar Refining
Spreckles Sugar, Manteca	1965	8'-6 × 6H	Beet Sugar Refining
Corn Sweeteners	1966	7' × 8H	Corn Syrup Refining
Industrial Sugar, Missouri	1966	6' × 8H	Cane Sugar Refining
Spreckles Sugar, Maryland	1966	6' × 8H	Cane Sugar Refining
West Virginia WW, Nitro, W. Va.	1966	6' × 8H	Water Purification
Penick & Ford Ltd., Iowa	1967	30" × 6H	Corn Syrup Refining
The Upjohn Co., Michigan	1967	6' × 8H	Pharmaceuticals
Cargill Inc., Iowa	1968	7' × 8H	Glucose
Dow Badische, Texas	1968	30" × 6H	Acetic Acid
Dimmit Wheat Growers, Texas	1969	7' × 6H	Corn Syrup Refining
Holleytex Carpet Mills, Pa.	1969	30" × 6H	Dye Wastewater
Rhodia Inc., Oregon	1969	6' × 6H	Chlorinated Phenols
C.P.C. International, Yonkers, N.Y. (Refined Syrups)	1970	10' × 8H	Cane Sugar Refining
Penick & Ford, Iowa	1970	12' × 6H	Corn Syrup Refining
St. Lawrence Starch, Canada	1970	6' × 5H	Corn Syrup Refining
BP Oil, New Jersey	1971	6' × 6H	Wastewater
Eli Lilly	1971	6' × 6H	Pharmaceuticals
FMC Corp., Wyoming	1971	7' × 6H	Soda Ash
Pfizer Chemical, Connecticut	1971	8'-6 × 6H	Citric Acid
Stepan Chemical Co., N.J.	1971	6' × 6H	Wastewater
Hercules, Mississippi	1972	12' × 6H	Wastewater
Reichhold Chemicals, Alabama	1972	13'-6 × 6H	Wastewater
South Coast Corp.,	1972	10' × 6H	Cane Sugar Refining
Amerada Hess, New Jersey	1973	8'-6 × 6H	Wastewater
American Aniline, Pennsylvania	1973	6' × 6H	Wastewater
Anheuser Busch	1973	10' × 6H	Corn Syrup Refining
Esso Research	1973	30" × 4H	Wastewater
St. Lawrence Starch, Canada	1973	6' × 4H	Corn Syrup Refining
American Maize Products	1974	10' × 6H + OH	Corn Syrup Refining
Cargill Inc., Dayton, Ohio	1974	8'-6 × 7H	Corn Syrup Refining

Table B

Industrial Carbon Regeneration Furnace List

User	Date of Installation	Size	Application
Cargill, Inc., Memphis, Tenn.	1974	8'-6 × 7H	Corn Syrup Refining
Corn Sweeteners	1974	13'-6" × 6H	Corn Syrup Refining
Onisa, Mexico	1974	7'-0" OD × 6H	Cane Sugar Regeneration
Republic Steel Corp.	1974	16'-0 × 8H + OH	Wastewater
Corn Sweeteners	1975	13'-6" × 6H	Corn Syrup Refining
Imperial Sugar Co., Sugar Land, Tx.	1975	12'-9" × 5H + OH	Cane Sugar Refining
American Maize Products, Decatur, Al.	1975	12'-9" × 6H	Corn Syrup Refining
Clinton Corn, Montezuma, N.Y.	1975	14'-3" × 6H	Corn Syrup Refining
Cargill, Inc., Dayton, Ohio	1975	8'-6" × 6 + OH	Corn Syrup Refining
Corn Sweeteners, Cedar Rapids	1976	13'-6" OD × 8H	Corn Syrup Refining

Viral Removal

While viral inactivation is well documented with chlorination usage, lime treatment of secondary sand-filtered effluents has been shown to virtually eliminate viral problems. Figure 2 shows that with lime treatment applied to a secondary effluent to arrive at a pH of 11.1, inactivation of polio virus was virtually complete within 90 minutes. Table IX contains data from a 1969 sample for South Lake Tahoe effluent, illustrating removal of viral components. The one exception that occurred was eliminated after the effluent passed through the carbon adsorption and chlorination steps whereby the final effluent was found to be free of all remaining viruses.

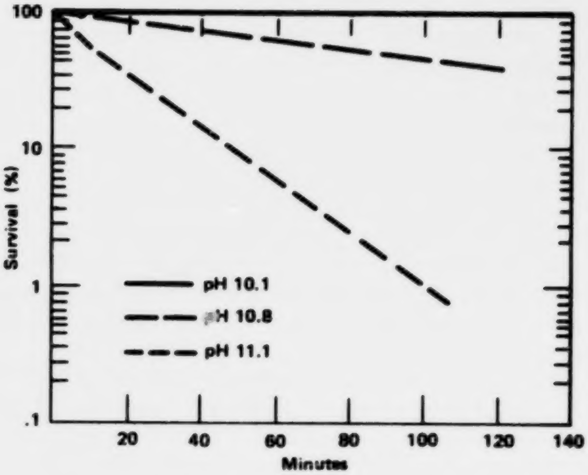

Figure 2. **Inactivation of Poliovirus 1 by high pH at 25°C in Lime-Treated (500 mg/l Ca (OH)2), sand-filtered secondary effluents.**

Table 9

Virus Sampling, 1969, at the South Tahoe Public Utility District Water Reclamation Plant

Viruses Recovered (Plaque Forming Units)

	Primary Effluent	Secondary Effluent	Carbon Column Unchlorinated Effluent	Chlorinated Final Effluent
May 29	3	0	1	0
June 5			0	0
June 12			0	0
Aug. 20	3	18	NRD*	0
Aug. 27			NRD*	0
Sept. 11			0	0
Sept. 18	179	14	9	0
Sept. 25	NRD	430	0	0
Oct. 2	207	320	0	0

*No reliable data.

Human Health Data

As previously discussed, the problems of trace organics, toxic metals and viral impurities are virtually eliminated in advanced wastewater treatment facilities that utilize carbon filters and lime treatment. The addition of chlorination can be utilized effectively to enhance removal of these constituents in addition to being an excellent unit process to eliminate bacterial pathogens.

Tertiary quality treatment systems deliver wastewater effluents that are equal to or surpass the quality of many current water supplies in the U.S. However, such treated effluents often have some trace impurities (organics, metals, etc.) as do fresh potable water sources. How important or critical from a health standpoint are these substances? Although we do not presently know the answer to this question, some research on this subject is underway.

It is useful to evaluate human experience as an additional assurance of safe drinking water quality. Data of this nature are scarce. We can look to Windhoek for further information on the health implication of drinking reclaimed water. Direct use of reclaimed drinking water has occurred in the Windhoek, Namibia, wastewater treatment plant which is the only advanced wastewater treatment installation in the world that directly integrates purified wastewater into the city's domestic water supply on a continual basis. The facility played a vital role in alleviating the water shortages during the acute drought of 1969-1971. This installation consists of a conventional wastewater treatment plant which had algae lagoon and physical-chemical treatment stages added. The effluent met the United Nations World Health Organization (WHO) drinking water standards and was used to supplement up to 50% of the tap water for this community of approximately 70,000 people (3, 15).

It turns out that extensive testing has shown that viruses were removed from the effluent (16). During 1976, studies at Daspoort also confirmed the Windhoek sampling data. The data demonstrated that viruses and bacteriological contaminants were present in treatment plant influent, but absent from the treated effluent.

Epidemiologic studies analyzing the long-term health consequences of trace organics and other trace residuals in the tertiary treated wastewater of the Windhoek municipal population were unable to detect any change in disease or morbidity patterns that could be associated with the consumption of tertiary wastewater. While more epidemiologic data in the U.S. and other countries are needed to provide better assurances of safety, the Windhoek work is an initial positive data point indicating a probable lack of adverse chronic health effects associated with consumption of high quality reclaimed wastewater.

Summary

The parameters of greatest health concern with direct reuse of reclaimed water are vectors of a viral, organic chemical and inorganic chemical or trace metallic nature. The biological or infectious agents are effectively eliminated by modern advanced sewage treatment methods. The combination of lime treatment and carbon adsorption have been shown to effectively eliminate organic compounds, toxic metals and viruses from advanced wastewater effluents. Unit processes such as ozonation, carbon regeneration, chlorination plus many others can reduce costs and improve pollutant removal. Human experience is scarce, but the information available suggests that modern technology for wastewater reuse is acceptable in terms of public health.

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Aftertreatment of Biochemically Treated Wastewaters before Their Reuse

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A method for biochemical treatment of industrial and storm refinery wastewaters returned to the circulating cooling water systems has been widely applied over the recent years. To biochemically treat these wastewaters, one-stage complete-mix activated sludge systems providing for 6 to 8-hr aeration are largely used. Quality of wastewaters before and after biochemical treatment is shown in Table 1.

Table 1

Quality of Wastewaters Before and After Biological Treatment

No.	Analysis	Before treatment mg/l	After treatment mg/l
1.	Oil Products (extractable with hexane)	up to 25	3-5
2.	COD	-"- 400	70-100
3.	BOD_5	-"- 150	10-15
4.	BOD_{total}	-"- 250	15-20
5.	Sulfides	-"- 20	abs.
6.	Phenol	-"- 10	0.1
7.	Ammonium nitrogen	-"- 30	4-8
8.	Suspended solids	-"- 50	10-90
9.	pH	6-8	6-8

As is seen from the Table, biochemically treated wastewaters are essentially free from contaminants. Their chemical composition is in full accordance with quality standards for make-up water with the exception of suspended solids.

The wastage of activated sludge from activated sludge clarifiers remains larger and in some cases (technological regime deviations) may be in excess of 100 mg/l. The activated sludge together with the activated sludge effluent goes to coolers where it settles on cooling surfaces and reduces the heat transfer factor. In the case of 1-mm deposits on tubes, the heat transfer factor is reduced by 10 to 30%.

Thus, the main task is to remove suspended solids from biochemically treated wastewaters before their return to a circulating system.

One of the promising treatment methods is filtration.

The applicability of granular and pressure screen filters has been tested. Quartz sand of a 0.5 to 2mm size (equivalent diameter 1.25 mm) and coke of a 1.5 to 3.5 mm size (equivalent diameter 2,3 mm) have been tested as a granular bed. The filter bed height is 1 m. According to the USSR standards, filtering media should meet the following requirements for mechanical strength: grindability — max. 4%, attrition — max. 0.5%. Quartz sand used in filters has a grindability of 3.6% and an attrition of 0.14%. Petroleum delayed coke used as filter bed has a grindability of 1.14% and an attrition of 0.4%, which shows its applicability.

Tests were performed on pilot filters with an efficiency as high as 100 l/hr. Sand was used in one filter, coke — in the other and then both filters were operated in parallel with the same influent. The average test results are presented in Table 2. One can see from the reported data that the best treatment quality and the best filter run duration are achieved at an average filtration rate of 8 m/hr.

Such a difference in filter run duration is attributed to the filter bed size composition. Coarse grained filters which provide optimum filter performance are needed for aftertreatment of biochemically treated wastewaters.

It should be noted that when operating pilot filters, their regeneration was due to the achievement of maximum pressure loss, which was indicative of a strong adherence of the retained activated sludge to the filter medium, the filtrate quality remaining good.

Therefore, while regenerating filters, a prior air loosening-up of the filter bed is used, which results in the destruction of contaminant clots due to the intense intermixing of filter bed and the removal of contaminants from the surface of particles. Air for loosening-up is supplied by an air blower. It is fed to a separate air distribution system located in the lower filter part. Before feeding air into the distribution system the water

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Table 2

Performance of Granular Filters

No.	Average filtration rate, m/hr	Pressure loss, mm Hg initial	final	Filter Run duration, hr	Suspended solids, mg/l influent	filtrate	Filter bed
		Pilot filter					
1.	7.9	35	650	12	62	3.3	Sand, 0.5-2.0mm
2.	8.1	40	650	10	81	4.0	-"-
3.	8.8	17	657	18	118	5.2	Coke, 1.5-3.5mm
4.	8.1	19	650	22	79	4.7	-"-
		Industrial filter					
5.	8.0	45	620	9	35	6.5	Sand, 0.5-2.0mm
6.	7.2	48	600	11	38	6.0	-"-
7.	7.3	52	600	10	44	8.0	-"-

level in the filter is lowered by 0.3 to 0.5 m in order to prevent the filter medium wastage. Air blowing is carried out during 5 minutes with an intensity of 13 to 15 l/sec·m^2. After completing the air blowing a conventional back washing of filter bed is performed. Water is fed to a drain distribution system located under the air system. Washing intensity is 18 l/sec·m^2, washing duration is 10 minutes. Washings are sent to aerotanks. Washing water consumption amounts to 2-3% of the quantity of filtered water.

It should be noted that in spite of the use of air for filter regeneration the filter bed is loaded with time by very dense residual contaminants. They are essentially presented by biogrowths that strongly adhere to the filter bed, which necessitates a periodic treatment of filter bed with biocides.

Chlorine water containing 100 to 200 mg/l of active chlorine is recommended as biocide. To prepare chlorine water, typical chlorinators are used. Filtering media are treated after the achievement of a residual pressure loss of 0.3 to 0.5 m for pressure filters and 0.2 to 0.3 m for open filters. Before treating filter with chlorine water conventional washing is performed. Water is then fully drained from the filter and the latter is filled with chlorine water. After a 1-day contact of chlorine water with filter bed the former is drained to a separate vessel where, if necessary, it is neutralized by sodium thiosulfate and sodium carbonate.

After the drainage of chlorine water the filter undergoes a conventional washing during 2 to 3 minutes, the first portions of washings being discharged to sludge tanks.

The averaged data obtained on industrial filter with quartz sand filter bed of a 0.5 to 2.0 mm size composition are presented in Table 2. One can see from the data that at a filtration rate of 7.2 to 8 m/hr the filter provides a sufficiently high quality of wastewater treatment at a level of 6 to 8 mg/l. Water of this quality is in full accordance with the requirements for water returned to a circulating cooling water system.

Highly efficient self-cleaning VSF type ("VSF" means "filter with a high filtration rate") filters of a serial production are destined for mechanical water treatment on screens to remove suspended matter. The filters are applied in chemical, food, paper, metallurgical, and other industries.

Filtration of biochemically treated wastewaters has been tested under industrial and laboratory conditions using these filters. VSF-2000 filter (Fig. 1) comprises the following main parts:

- cylindrical shell;
- two filtering elements;
- washing device;
- drive.

The filter is completed with a control board.

Maximum filter efficiency, m^3/hr	2000
Electric power consumption, kwh	0.5
Installed capacity of electric motor, kw	1.7
Overall dimensions mm	
diameter	2032
Weight, kg	3000

Water treatment is performed at a pressure of 10 kgf/cm^2 (1.0 M Pa). The washing device is switched on automatically, washing is carried out simultaneously with filtration and, depending on the quality of water to be treated, is performed continuously or periodically. This provides for a continuous filter performance.

The filter laboratory model (Fig. 2) 0 150 mm is a part of the filtering element of industrial filter. Investigations on filtration of biochemically treated wastewaters were performed at a pressure of 10 kgf/cm^2, at a filtration rate of 150 to 600 m/hr. Back washing was carried out using river and filtered water.

Based on the laboratory filter model, pressure loss and filter run duration versus filtration rate as well as screen cleaning quality versus washing intensity and duration were studied.

Pressure loss is plotted against filtration duration in Fig. 3,4. As is seen from the curves the filter run duration is 7 to 24 hours for PO-0.25 -2 screens packet (mesh

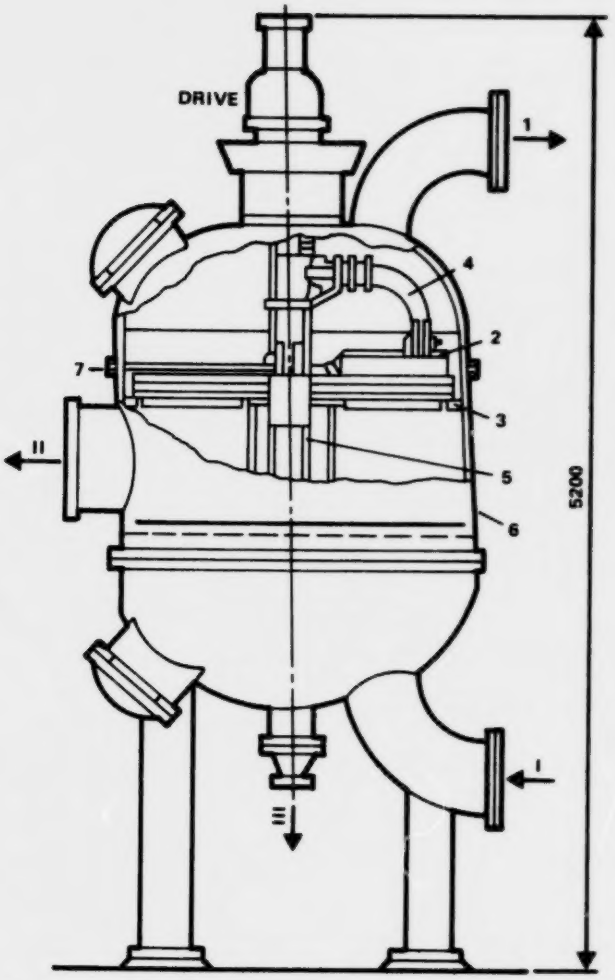

Fig. 1.
VSF-2000

size of 200 to 250 micrometers) and 10 to 80 minutes for S -200 dense screen, its value being decreased with increasing filtration rate. The curves convexity may evidence the carry-over of contaminants from screens.

Activated sludge settled on the screens has a density of 1.12 ÷ 1.15 and an ash content of about 25%. The deposit can be considered to be compressible. Pressurized particles or their aggregates are deformed, which leads to an increase in the deposit resistance with increasing pressure difference during the filtration process.

The back washing efficiency of screens for the purpose of their cleaning was evaluated by initial pressure drop and filtration run duration subsequent to the washing. The obtained data are presented in Fig. 5. As is seen from Fig. 5, an increase in washing intensity up to 400 $l/m^2 \cdot sec$. leads to a decrease in initial pressure drop on the screen and accordingly to an increase in filter run duration. A further increase in intensity does not improve the screen cleaning degree. The removal of residual contaminants requires a prior treatment of screens to lessen the forces of adhesion of activated sludge to the screen.

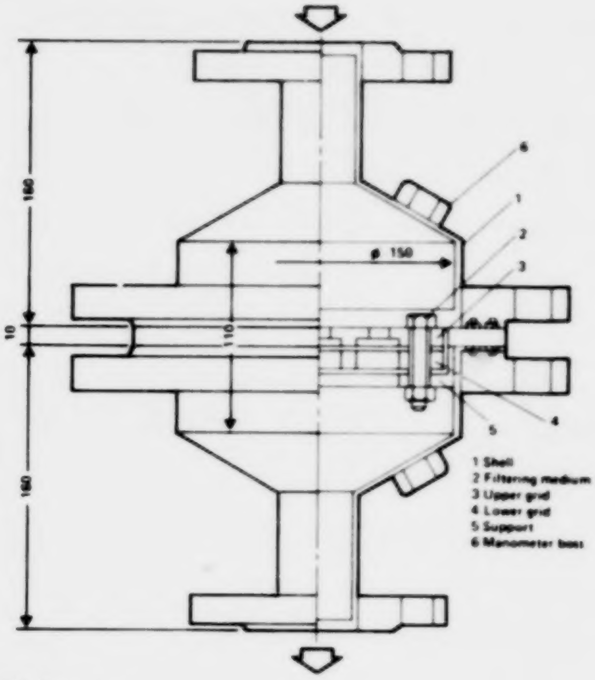

Fig. 2.
VSF filter model

As with filtration using granular filter beds, the forces of sludge adhesion to the screen are lessened under the action of biocides. A one-day treatment of screens using chlorine water with a concentration (based on active chlorine) as high as 200 mg/l makes it possible to clean screens by a subsequent back washing. This is confirmed by an experience of operating VSF-2000 industrial filter in one of the petroleum refineries.

Washing time variations over the range of 5 to 30 seconds did not affect the washing quality. This time seems to exceed the required one; the time necessary for washing one section of VSF filter is 1 to 2 sec.

In the performed experiments, the average effect of purifying biochemically treated wastewaters was not more than 10 to 15%; for a number of samples the content of suspended solids in filtrate was higher than that in influent, especially at high filtration rates such as 400 to 600 m/hr, which is attributable to the carry-over of contaminants from filter screens under these conditions.

Along with tests on the filter model, the VSF-200 industrial filter (used for aftertreatment of biochemically treated wastewaters before their return to the circulating system) has been studied. The filter has been mounted on the open site and operated automatically. The filter flow diagram is presented in Fig. 6.

Effluents from activated sludge clarifiers are partially passed through the filter and fed as make-up water to circulating systems of the refinery. The filter washing water is pumped to aerotanks.

During the initial operating period the filter run duration amounted to 3 hours. It is gradually diminished with time to 20-30 minutes. The filter is switched off.

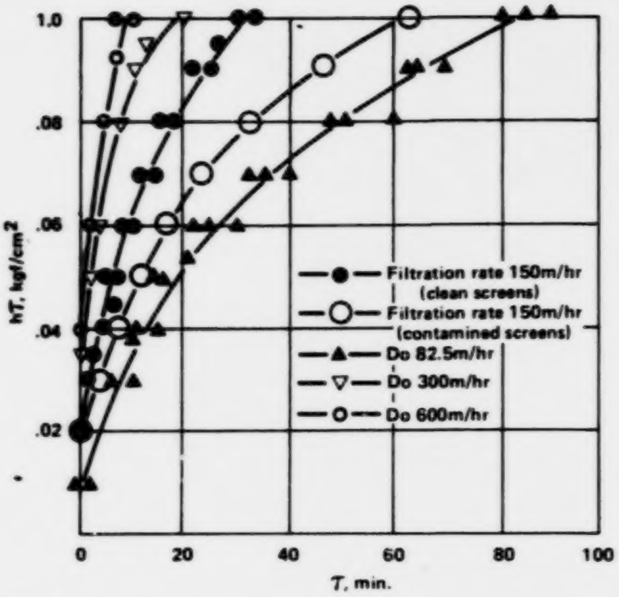

Fig. 3.
Pressure drop versus filtration time (S-200 screen)

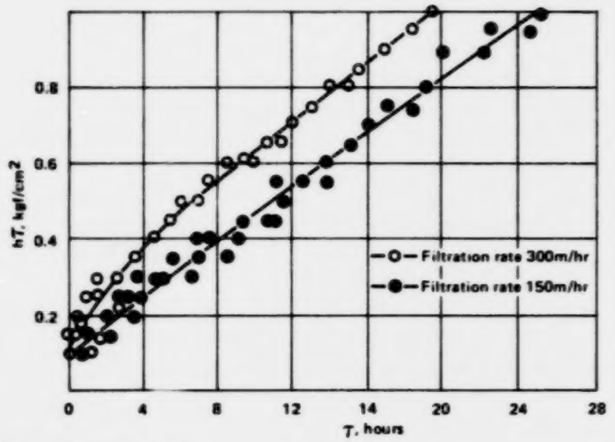

Fig. 4.
Pressure drop versus filtration time (PO - 025 + 02 packet)

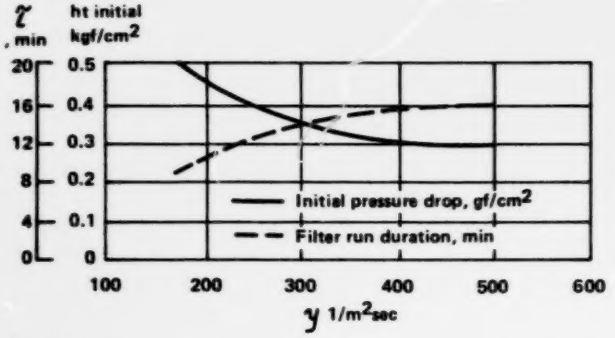

Fig. 5.
Initial pressure drop and filter run duration versus washing intensity (S-2000 screen)

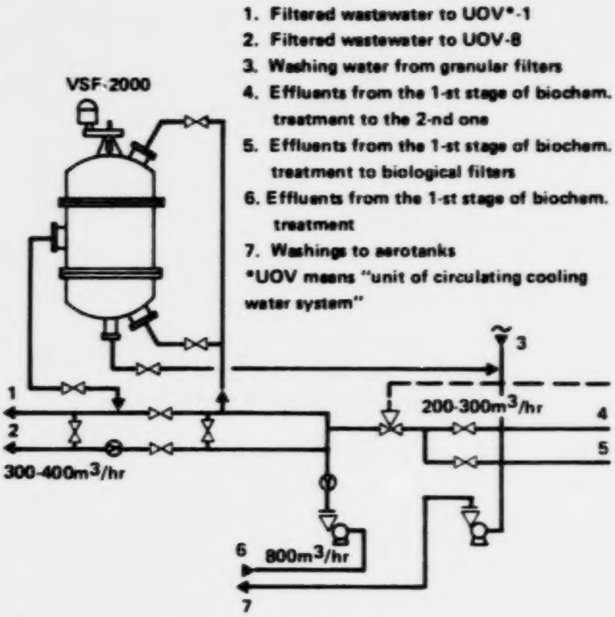
1. Filtered wastewater to UOV*-1
2. Filtered wastewater to UOV-8
3. Washing water from granular filters
4. Effluents from the 1-st stage of biochem. treatment to the 2-nd one
5. Effluents from the 1-st stage of biochem. treatment to biological filters
6. Effluents from the 1-st stage of biochem. treatment
7. Washings to aerotanks
*UOV means "unit of circulating cooling water system"
VSF-2000
1
2
3
4
5
6
7
300-400m3/hr
200-300m3/hr
800m3/hr

Fig. 6.
Flow diagram of VSF-2000

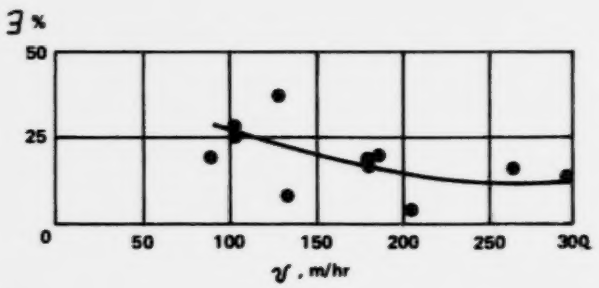

Fig. 7.
Purification effect versus filtration rate

The screens are treated with chlorine water during 24 hours 1-2 times per month.

The average purification effect of the filter is 20% at 50 mg/l suspended matters in the influent. The filtration rate ranges from 100 to 300 m/hr.

Averaged data showing the purification efficiency for different concentrations of suspended matters in the influent are presented in Table 3. An increase in the content of contaminants in the influent leads to an increase in their residual content in the filtrate; the purification effect becomes somewhat higher but the residual contamination of filtered water remains high.

Fig. 7 shows purification effect versus filtration rate. The diagram is based on the operating data for VSF-2000 industrial filter over a long period. As is seen from Fig. 7, a decrease in filtration rate leads to an increase in purification effect but the latter does not exceed 30-40%. At a filtration rate of more than 200 m/hr the purification effect is 10 to 15%.

The performance comparison of granular and screen filters shows that in spite of compactness and high water filtration rates in the case of screen filters, granular filters provide a higher aftertreatment quality of wastewaters because of their higher efficiency.

Table 3

Performance of VSF-2000 Filters

(averaged data)

	Ranges of initial contamination mg/l			
	10-25	26-50	51-100	101-105
Initial content of suspended matters, mg/l	21.7	38.2	56.2	117.4
Residual content of suspended matters, mg/l	18.0	31.7	42	85.5
Purification effect, %	17.7	17	25	27

Facilities for the Treatment of Biologically Treated Effluents from Industrial and Municipal Sources by Ozone**

by

William J. Lacy, P.E.*

Background

Ozone is a colorless gas naturally occurring in the atmosphere. It is a particularly active form of, and in fact is produced from, and decomposes to, the element oxygen. Its molecule contains three oxygen atoms, whereas the more common form, which comprises about a fifth of the air we breathe, has only two oxygen atoms per molecule. Ozone results from exposing oxygen to energy such as from electrical discharge or irradiation. Its activity results from the release of this energy as the molecules resume their original state. This decomposition takes place rather rapidly, so that the energy may be harnessed for a number of useful purposes. Because of this, ozone is frequently called "activated oxygen."

It exists naturally at relatively high concentrations in the atmosphere at an altitute of about 20,000 to 30,000 meters. This ozone layer in the upper atmosphere is crucial to terrestrial life because it shields the earth from harmful ultraviolet light radiations.

At the earth's surface, the concentration of ozone is greatly reduced. Terrestrial ozone occurs naturally as a result of reactions in the atmosphere and that which may filter down from the upper levels. It is not, as are many air pollutants, present as a result of synthetic products from chemical sources. The slightly pungent traces of odor in the air during and following a thunderstorm are due to ozone.

Ozone is one of the most powerful oxidants available to man. It is an effective bleach. It is a powerful sterilant, killing bacteria and fungi more rapidly than chlorine. It also attacks viruses and carcinogenic materials that are not attacked by those chemicals most often used in treating water. It deodorizes by destroying odor-causing substances rather than by masking the odor.

The commercial use of ozone for purifying drinking water is over seventy years old. Its major application has been in regions where raw water supplies are of generally poor quality and a method of advanced treatment of drinking water, such as ozonization, is required to produce economically an esthetically pleasing supply free of pathogens.

As problems of pollution, safety, and rising costs in all the industrial areas concerned with treatment of water, wastes and air become more difficult to handle, ozone is providing an answer from both technological and economical considerations.

Ozone must be generated on-site near its point of use. Ozone cannot be stored so only the amount being generated at any instant might be accidentally released. This eliminates all of those safety problems associated with transportation and storage of chemicals. Because of its unique nature it is easily identified, and generation may be immediately ceased on detection of its presence. At higher, potentially toxic levels, it rapidly degrades into oxygen.

Ozone is not an environmental panacea, but the continuous development of its valuable attributes as a viricide, general disinfectant and pollution-free oxidant should be utilized by informed scientists and engineers.

Introduction

Historically, sewage treatment plant effluents in the United States have been disinfected with chlorine. During the late 1960's it began to be apparent that discharges of chlorinated sewage treatment plant effluents to rivers, lakes and streams were sometimes causing severe environmentally detrimental effects upon the aquatic lives in these receiving bodies of water.

In November, 1974, EPA made public results showing that treating drinking water supplies with chlorine produced many halogenated compounds, some of which

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**Presentation by Mr. Harold P. Cahill, Director, Municipal Construction Division, EPA, at the US/USSR "Facilities for Tertiary Treatment of Biologically Treated Effluents" Symposium in VODGEO, Moscow, USSR, November 13-15, 1978.

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might be responsible for causing liver carcinomas. The U.S. Environmental Protection Agency, Office of Research & Development, funded a grant to the International Ozone Institute to conduct a state-of-the-art review of the published literature dealing with the use of ozone. About 240 published articles have been reviewed. Abstracts of each article appear in the final report. (1) The discharge of industrial wastewaters is being regulated by EPA by the Water Pollution Control Act Amendments of 1972. The objective of WPCA is to halt the promiscuous discharge of pollutants by applying wastewater treatment techniques commercially available in 1977, and even more advanced technologies to produce effluents of improved quality by 1983.

Progressively, the quality of industrial discharges will improve. Industry is encouraged to recycle and/or reuse as much of its wastewater as posssible and many manufacturers presently are studying the use of ozone, a powerful oxidant, to destroy pollutants and to otherise purify certain wastewater.

Municipal Wastewater Treatment

I. *Disinfection*

A. EPA Policy

In the United States, until recently disinfection of sewage treatment plant effluents has been required by the EPA. The original concept was to disinfect to protect the public health.

The continued discharge of tons/day of chlorine into the environment has been shown to cause massive fish kills, as well as to produce chlorinated organic materials which can progress up the food chain. EPA has recently (1) enunciated a policy that disinfection (currently synonymous with chlorination—in the United States) need only be practiced where known to be necessary to protect the public health. Estimates equate this to mean that at least 80% of current municipal effluents will continue to have to be disinfected. All plants will have to have standby disinfection facilities, in case of emergenices.

EPA's role is to protect the public health—first by insuring that disinfection is practiced where it is needed and, secondly, by demonstrating to states, local municipalities and their consulting engineers the benefits, drawbacks and economics of methods which are viable alternatives to chlorination.

Two types of ozone contactors were evaluated with respect to performance, coliform reduction and ozone efficiency. The two systems included a packed column filled with ceramic media and a jet scrubber unit. It appears that a single-stage jet scruber is not a feasible gas-liquid contactor for effluent coliform reductiion with ozone. The addition of one or more jet scrubbers in series would probably improve coliform reduction and utilization, but the needed repressurization costs would likely offset any savings in increased efficiency.

An EPA demonstratioin grant was awarded to the city of Wyoming, Michigan, to compare the toxicities of sewage treatment plant effluents on certain species of fish. Side-by-side comparisons of the effects of chlorination, chlorination/dechlorination, ozonation, bromine chloride and no disinfection were conducted on two effluents—one largely municipal sewage and the second containing about 50% industrial wastes.

Results of this demonstrated program (2) reveal that ozone is effective as a disinfectant, although at higher cost than gaseous chlorination. In addition, ozonated effluents are not toxic to indigenous aquatic life.

Several cities in the USA have been considering changing to ozonation for some time, and the first operational plant to install ozone for this purpose was at Indiantown, Florida (3). This is about 2,000 cu. meters/day sewage treatment plant which incorporates an Imhoff tank, trickling filter, chemical coagulation, and settlings of solids, followed by ozonation. The plant has been fully operational since November, 1975, and the city of Indiantown is pleased with the results. See Figure 1.

Figure 1

U.S.A. Sewage Treatment Plants Using Ozone (1978)

Location	Size (mgd)	Purpose	Startup Date
Woodlands, TX	0.6	Disinfection	1975
Indiantown, FL	0.5	Disinfection	1976
Estes Park, CO	3	Disinfection	1977
Springfield, MO	35*	Disinfection	1977
Orange County, NY	2	Disinfection + Dissolved Oxygen	1977
Santa Clara, CA	4	Disinfection + Organics	1977
Under Construction			
Meander Creek, OH	5*	Disinfection	2/78
Chino Basin, CA	3	Susp. Solids	early 1978
Potomac Heights, MD	0.2	Disinfection	early 1978
Concord, NC	30*	Disinfection	late 1978
Pensacola, FL	24*	Disinfection	late 1978
Murphreesboro, TN	10*	Disinfection	9/78
Frankfort, KY	9	Disinfection	bid 12/77
Madisonville, KY	4.5*	Disinfection	bid 12/77
Under Design			
Cleveland, OH	50-100**	Organics + Disinfection	1980
Indianapolis, IN	2 × 125*	Disinfection	bid late 1978

at least 15 others known, 12 oxygen activated sludge

*Oxygen Activated Sludge Process

**Ozone before and after activated carbon

Woodlands, Texas (2,300 cu. meters/day) and Estes Park, Colorado (11,400 cu. meters/day) have recently begun operation, and ozonation for disinfection is incorporated in these plants. During 1977, *six* additional U.S. cities began operation of sewage treatment plants using ozone for disinfection. Three other plants have been bid and at least 15 other plants are known to be in the design stages at this time. The growth of ozone use as an effluent disinfectant was recently discussed in the May 11, 1978 issue of *Engineering News Record* (7).

B. *Oxygen Activated Sludge Process*

Ozone technologists recognize that about half the electrical energy is required to generate a given weight of ozone from oxygen than from air. These favorable economics can be coupled with sewage treatment processes which employ oxygen activated sludge processes. See Figure 2. After generating ozone from oxygen (and producing 2% ozone in oxygen) then disinfecting the activated-sludge effluent, the contactor off-gases (still very rich in oxygen) can be sent to the biological reactor. Then most of the oxygen originally taken for ozone generation can be utilized in the treatment process. The only oxygen "lost" will be that which actually dissolves in the disinfected effluent.

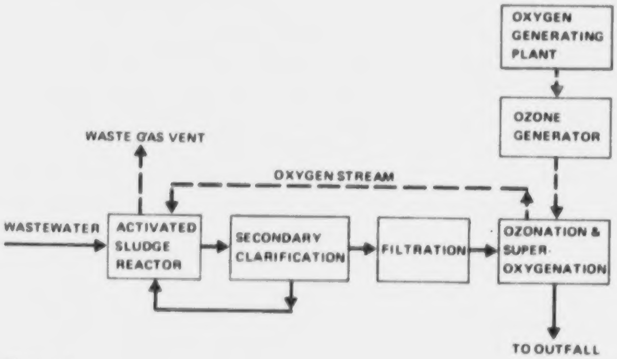


Figure 2
The Oxygen Activated Sludge Process

Even this "lost" oxygen is utilized, because it serves to increase the dissolved oxygen content of the receiving body of water, which is beneficial to the local aquatic life.

The first USA sewage treatment plant to couple ozone generation from oxygen with the oxygen activated sludge process was a 22,800 cu. meters/day plant at Meander Creek (near Youngstown, Ohio). (5), (6). This plant is now in the early stages of start-up. One other oxygen activated sludge plant using ozone disinfectant is operational. The March, 1978 *Water and Wastes Engineering* describes the ozonation expanded wastewater plant in Springfield, Missouri (8).

II. *Suspended Solids Removal*

The State of California has adopted very stringent discharge standards, particularly with respect total coliforms, viruses and dissolved solids. For example, disinfection standards for the more water-rich areas of the USA are normallly 200 fecal coliforms/100 ml (most probable number); however, the standard in California is 2.2 *total coliforms*/100 ml.

Chino Basin, California is constructing about 12,000 cu. meters/day activated sludge sewage treatment plant. To meet the suspended solids standard by standard treatment techniques requires the addition of a chemical flocculent (alum) followed by filtration. Since flocculents are soluble chemicals, their use will result in increased levels of dissolved solids in the effluents, such that the California Total Dissolved Solid (TDS) standard now would exceed. Filtration of chemically flocculated effluents is relatively slow, and the chemical sludge must be disposed of. To reduce TDS to an acceptable level would require additional effluent processing, thereby adding costs.

Engineers for Chino Basin (4) discovered that by applying 10 mg/l of ozone directly to the secondary effluent the suspended solids are coagulated, filtration is rapid, the TDS remains below the level required, and there is no chemical sludge to be disposed of.

This novel application for ozone in sewage treatment was incorporated into the plant design, construction is in its later stages, and the plant is expected to begin full-scale operation this year.

Industrial Wastewater Treatment

I. *Cyanides*

Destruction of cyanides by means of ozone takes place in two discrete steps, the first being rapid oxidation to cyanate, followed by the much slower oxidation hydrolysis of cyanate to carbon dioxide and nitrogen.

Iron cyanide complexes are very stable to destruction with ozone alone, the cyanide portion being only slowly decomposed. However, in combination with ultraviolet radiation, destruction of the complex and of the cyanide is very rapid. In effecting a more rapid destruction of such stable complexes, the ozone/ultraviolet combination conserves energy, since otherwise very long treatment times would have to be employed.

Laboratory studies of cyanide containing simulated gold mining wastewaters were conducted by the Canadian Department of Energy and Mines, using the Film Layer Purifying Chamber (a spray tower type of ozone contactor). In this contacting apparatus, cyanides apparently were converted totally to carbon dioxide and nitrogen in a contact time of about 30 seconds. Neither cyanide nor cyanate were detected in the ozonized effluent (9).

Since the mid-1950's, ozone has been used by the Boeing Company, Wichita, Kansas, to remove cyanides from their metal finishing wastes (10), (11). Combined wastewater streams from various metal finishing baths are ozoned prior to discharge into a local receiving lake. This plant has met all applicable discharge standards for cyanide consistently over the past 20 years. Recently, the company announced that it will expand and upgrade the capabilities of its ozonation treatment system.

EPA has funded a demonstration of ozonation to reduce cyanides in metal finishing wastewaters at Sealectro Corporation, Mamaroneck, New York (12). This plant has been operating successfully since early 1974, and all applicable local discharge standards are being met.

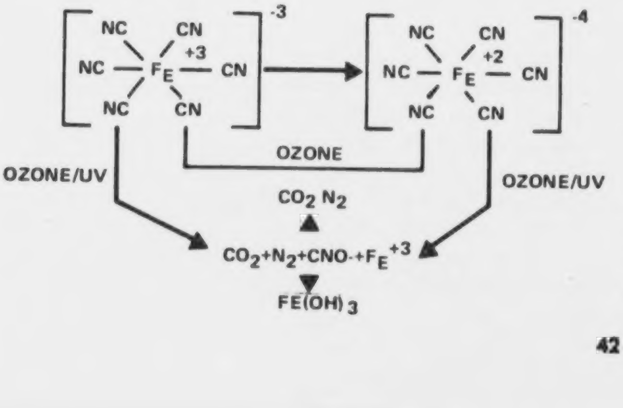
Since late 1976, Hughes Tool Company, Houston, Texas, has been treating its metal finishing wastewaters with ozone in synergistic combination with ultraviolet radiation. Wastewater flows of up to 80 liters/minute are treated first with ozone (to remove simple cyanides) and then with ozone/UV to remove otherwise stable iron complexes and heavy metals levels of 30-100 mg/1. This treatment system reduces total cyanide and heavy metals to levels of 0.1 mg/1 (13).

Photoprocessing Wastes

A novel use for ozone in the photoprocessing industry has been developed recently, which is the oxidative regeneration of bleaching compositions. Iron ferricyanide is used as the photoprocessing bleach, and in so bleaching, the ferric (trivalent) iron is reduced to the ferrous (divalent) state. Both iron cyanides complexes are so stable to ozonation that the ferrous iron is merely converted back to the ferric form upon ozonation, without degradation of cyanide moieties or destroying the complexes themselves.

This ability to regenerate iron cyanide complexes is being exploited commercially by a small U.S. company who manufactures an integrated ozonation treatment system specifically for regenerating spent photoprocessing laboratory wastes, the spent bleach being subjected to ozonation. During the night shift, however, when no bleach is being generated, the ozone generator is automatically programmed to treat the other more readily oxidizable components in photographic processing baths. Thiosulfate is converted to sulfate, hydroquinone and other organic compounds are destroyed. Thus the ozonation system is programmed to regenerate spent photographic processing bleach during the hours of laboratory operation, then it is programmed to treat other oxidizable components of the processing wastes at night (14).

Figure 3
Ozonization of Iron Cyanides



In this manner, the pollutional load of photoprocessing wastewaters can be reduced. It is claimed that the integrated ozone treatment system can pay for itself, on the basis of bleach recovered alone, within a period of 3-5 years, depending upon the size of the installation.

An EPA supported demonstration of the efficacy of ozone regeneration of spent photoprocessing bleach was successfully completed at Berkey Photo, Needham, Massachusetts, in 1974 (15) and this company, as well as many others throughout the United States, Japan, and in some countries in Europe, are using this treatment technique.

Polychlorinated Biphenyls (PCB's)

PCB's are quite toxic in very small quantities and are very stable to oxidation. In addition, they are not completely adsorbed onto activated carbon. They have been defined as toxic substances, pose very great public health hazards, and hence methods of assuring their absence in wastewater discharges from certain electrical motor manufacturing plants must be found.

Recent EPA sponsored studies 1977+ (16) with the Versar Company of Springfield, Virginia, have shown that the combination of ozone with ultraviolet radiation (UV) can reduce the levels of PCB's in industrial wastewaters to less than 0.01 microgram/liters, and in many cases to below detectable limits. Activated carbon adsorption can only reduce PCB level to 0.1 microg/1. Having compared treatment costs, efficiencies and commercial availability of ozone/UV systems, EPA has proposed that existing sources and new sources of PCB discharges be required to meet a standard of 0.01 microg/1 which can be attained by ozone/UV systems.

Thus, the combination of ozone/UV has been specified as "Best Practicable Control Technology" (BPCT) currently available for the PCB's (as well as for photoprocessing wastewaters).

Phenols

In Sarnia, Ontario, The BP Oil Refinery discharges itswastewaters, after treatment, into Lake Ontario. A rather complex biological treatment scheme has been practiced for the past 20 years, the final step of which is ozonation to remove phenolic compounds and trace amounts of other organics which are not removed by earlier treatment process steps (17), (18).

Power Plant Cooling Waters

Biological growth in power plant condensers reduces the efficiency of heat exchange. Present techniques of combating this, and in cleaning biofouled surfaces and structures, include the addition of rather high concentrations of chlorine to the intake and cooling waters. Because of the massive amounts of water currently used in once-through cooling systems, many tons of chlorine are being discharged hourly into the environment, with detrimental effects on aquatic life.

The Electric Power Research Institute has recently funded the Public Service Electric and Gas Company of New Jersey (PSEG/NJ) to conduct demonstrations of the efficacy of ozone in side-by-side comparisons with chlorine. These successful demonstrations were conducted during the sumer of 1977 at two PSEG/NJ power plants in New Jersey.

Associated Treatment Considerations

I. *Organic Oxidation Products*

One uncertainty currently delaying more rapid acceptance of ozonation are questions of what oxidation products are formed when organic materials in water are ozonized and what are the toxicities of these materials to man and to the environment. Although considerable research now is being conducted to answer these significant questions, it will be years before quantitative answers are obtained for all compounds which might be formed in all cases.

This general subject was discussed in detail at the November, 1976, Workshop on Ozone/Chlorine Dioxide Oxidation Products of Organic Materials, sponsored by the International Ozone Institute and EPA, in Cincinnati, Ohio. It was the consensus of those present that (a) ozonation of organic compounds produces organic oxidation products which generally are more biodegradable than their precursors, (b) there are some compounds formed upon ozonization which exhibit mutagenic activities, (c) chlorination of the same starting organic compounds produces oxidized and halogenated products, (d) a greater number of biologically active compounds are produced upon chlorination than upon ozonation. (19).

Electric Power Requirements

One other point that should be mentioned is the electric power requirements per 500 grams of ozone. Figure 5 shows the figures as based on ozone equipment already available on the market today (20).

At this point on our learning curve, intelligent water and wastewater treatment technologists must conclude that any chemical oxidant (chlorine, ozone, chlorine dioxide, bromine chloride, permanganate, etc.) should be employed with caution, and with as much knowledge about the composition of the system to be treated as possible.

Figure 4

Industrial Categories Reporting Use of Ozone*

Category	**No. of Articles Found** **Actual Wastes**	**Total**
Aquaculture	1	6
Breweries	0	8
Electroplating	7	17
Food & Kindred Products	4	5
Hospital Wastes	0	2
Iron & Steel	7	12
Leather Tanneries	0	4
Mining	4	5
Organic Chemicals	10	51
Paint & Varnish	2	2
Petroleum Refineries	8	16
Photoprocessing	4	7
Plastics & Synthetics	1	2
Pulp & Paper	7	24
Soaps & Detergents	2	5
Textiles	14	17
Cyanide-Containing Wastes	—	13
Phenolic-Containing Wastes	—	26
Totals	77	232

*1978

Figure 5

Power Requirements per 500 gram/Ozone*

Ozone generator with 2% ozone concentration from pre-dried air	8.20 kwh
Oil-less compressor	1.67
Heatless air dryer	0.125
Ozone turbine contactors	0.550
Control, relays, timers, etc.	.001
Closed loop cooling waters	.001
TOTAL POWER REQUIREMENTS	10.557
OR ABOUT	10.6 kwh

(The above figures are based on commercially available ozone equipment).

*Stopka, K. "Ozone Plant Improves Efficiency and Economy of Wastewater Treatment" — *Water and Sewage Works*, April 1978.

Conclusion

Ozone is not an environmental panacea but the continuous development of its valuable attributes as a viricide, general disinfectant, and pollution-free oxidant should be utilized and enhanced by informed scientists and engineers.

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Investigation of the Loading Regeneration of the Mixed-Media Filter with the Descending Particle Seize Distribution for the Advanced Wastewater Treatment.

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One of the most widely used methods of the advanced treatment of the biologically treated wastewaters is the filtration on quick filters. At present at the stage of deep wastewater treatment various structures of filters are used. The most perspective are the structures which have realized the principle of the filtration with the descending particle seize distribution and in which the appropriate conditions for the intensive washing are created. The multi-media filters and filters with the ascending water flow meet the first requirement, the structures which make it possible to create the intensive regime of the water-air washing meet the second requirement.

Out of the great experience of the VNII VODGEO it is clear that the mixed-media filter, developed in the VODGEO, meets the above mentioned requirements to the full. The mixed-media filter has 3 layers of loading: gravel-support, sand-gravel, gravel-frame. The frame gravel fraction is 40-60 mm, the quartz sand fraction is 0,8-1,0 mm, the general height of loading including the supporting layers is 2,3 m (1).

Table 1 shows the efficiency of the advanced wastewater treatment on the mixed-media filters, quartz and two-layer filters when the concentration of suspended substances in the given water variates from 10 to 50 mg/l and BOD total variates from 10 to 25 mg/l.

Table 1

The effect of the advanced wastewater treatment on the quartz, two-layer and mixed-media filters.

Type of filter	The reduction of the suspended substances in the filtrate, %	The reduction of the BOD total, %
Quartz	92,0	80,0-72,0
Two-layer	84,9	80,0-70,0
Mixed-media	89,0	79,0-72,0

The effect of the advanced treatment is practically the same but the length of the interregeneration period of the mixed-media filter is 1,5 times as much as of the two-layer filters and two times as much as of the quartz filters.

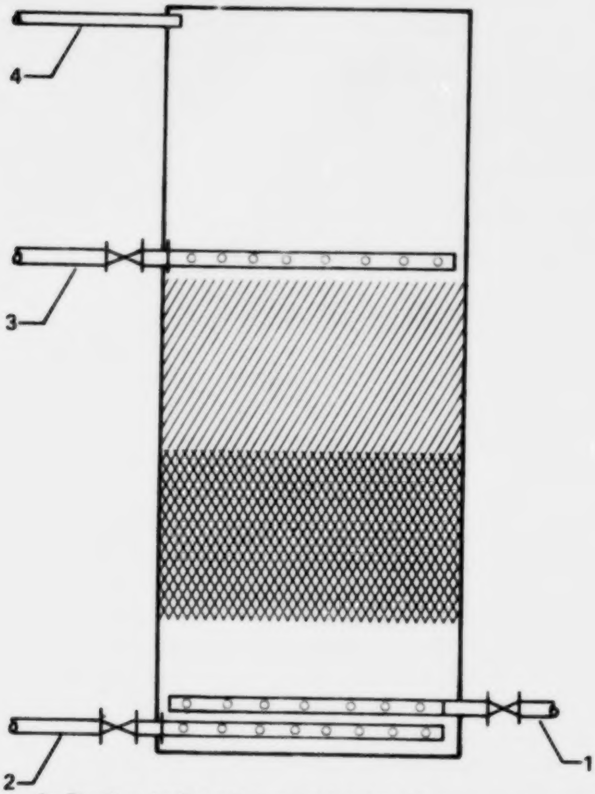
Mixed-media filters do not react to the variations of the wastewater quantity and to the variations of the wastewater pollution concentration. As a rule they work in the regime of the nonfilm filtration. The largely dispersed suspended substance of the activated sludge remains in the upper gravel layers of the filter, small-dispersed suspended substance remains in the low layers of the sand media.

Thus, mixed-media filter provides high efficiency of the advanced treatment of the biologically treated wastewater and to the full meets the requirement of the filtration with the descending particle seize.

To evaluate the conditions of the regeneration of the filter loading the investigations were carried out on the pilot plant mounted at the treatment facilities of the Salarskaya aeration station in Tashkent. There were 3 stages of the loading regeneration: I stage — air feeding, II stage — air and water feeding, III stage — water feeding. In the process of regeneration sand suspension occurs in the interpore gravel space and in the limited conditions of the ascending flow gravel and sand pollution is removed. The floaks of the washed sludge are transported by the washing water through the troughs, which are 200 mm higher than the loading surface.

Pic. 1 shows the principle scheme of the experimental mixed-media filter. The diameter is 1200 mm, height—4,8 m, capacity—250-300 m^3/day. In the course of the investigation isotope Au-198 was used. The definite volume of sand with fractions of 0,8-1,0 mm d = 0,9 mm was put into the isotope solution, then it was loaded to the filter on the surface of the sand media.

The coordination net-work was marked on the outside surface of the filter to determine the trajectory of the sand particle displacement in the process of washing. The absciss axis was located on the upper boundary of the sand layer. The points of the radioactivity measurement were marked at the net-work. As a result of the investigation the diagrams of isolines were obtained. They characterize the change of the radioactivity background before washing, in the process of washing and

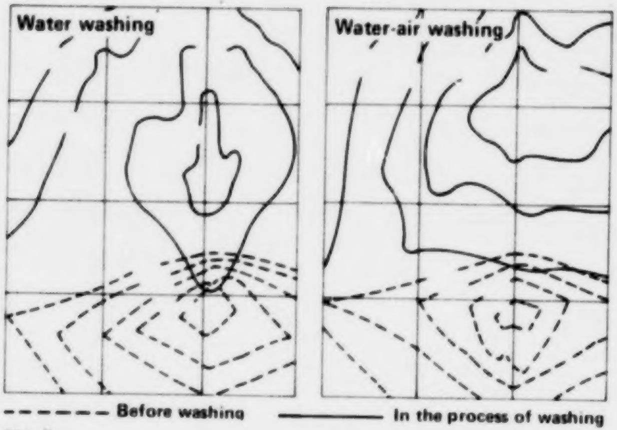


1– Pipe-line, feeding influent and draining polluted washing water;
2– Pipes for the feeding of the washing water and draining of the filtrate;
3– Pipe-line, feeding air for washing;
4– Overflow pipe-line.

Pic. 1
Principal scheme of the pilot mixed-media filter

after washing when the expenditure of water and air was different.

Pic. 2 shows two such diagrams, one for water washing, another — for water-air washing. In both cases the suspension of the sand media was provided by 100%. By the water washing the sand particles moved mostly in the vertical direction and after the water feeding ceased they occupied practically the primary position. By the water and air feeding thanks to the circulation flows the more intensive sand mixing and its displacement in the horizontal and vertical directions in the frame pores took place. After washing the marked sand, as a rule, moved to the deeper media. Providing in the mixed-media filter the intensive mixing conditions during the water-air washing doesn't create the danger of the gravel-support layer displacement, as the frame itself remains fixed. The trajectory of the sand movement during washing corresponds to the form of the isolines. The analysis of the diagrams shows that the conditions of the limited settlement created by the frame do not prevent the intensive mixing of the sand media. More than that, thanks to the limited space the speed of the ascending flow considerably increases. And because of this the possibility of the clash of the loading particles among each other and with the frame grains increases.



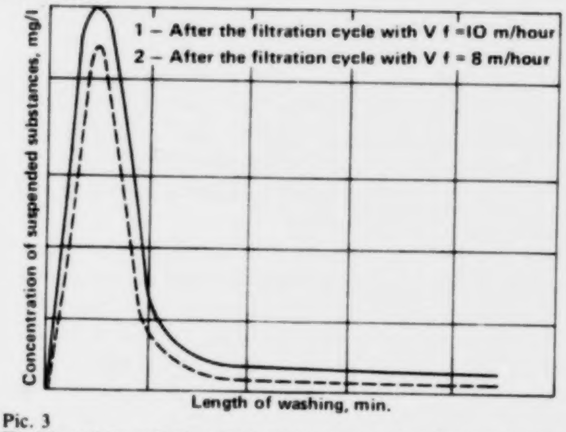
Pic. 2
Distribution of isolines of the radioactivity of the labeled loading in the course of washing

The investigations showed that the calculation intensity of the water feeding in case of water washing should amount to 20-24 l/sec m^2.

In case of water-air washing at the first stage air with the intensity of 14-16 l/sec m^2 is fed, at the second stage — air with the intensity of 14-16 l/sec m^2 and water with the intensity of 5-6 l/sec at the third stage — water with the intensity of 14-15 l/sec m^2.

The loading regeneration in such regime makes it possible to fulfil the washing in 8-9 minutes.

The diagrams on Pic. 3 show the quantity of the removed loading pollution using the recommended intensities.



Pic. 3
Washing of the pollution from the loading of the mixed-media filter

The main pollution mass is removed from filters at the 1 and 2 stages of washing. The low outlet of the washing water through troughs placed at the height of 200 mm above the loading level makes the transportation of the washed sludge floaks of every seize possible.

The resulted of the carried out investigations on loading regeneration of the mixed-media filter showed that these filters make it possible to use the intensive regime of the water-air washing, which provides the reliable removal of pollution.

Mixed-media filters are one of the most perspective structures for the advanced treatment of the biologically treated wastewater.

Municipal Wastewater Treatment by Land Application in the Minneapolis-St. Paul, Minnesota Area

by

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President
and
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Introduction

General

The Metropolitan Council of the Twin Cities is responsible for the preparation and adoption of a 20-year wastewater management plan for the Minneapolis-St. Paul Metropolitan Area, Minnesota. This plan is being prepared and funded under the auspices of Section 208 of the Federal Water Pollution Control Act (PL 92-500). The Metropolitan Council is the regional planning agency for a 7-county area including and surrounding the cities of Minneapolis and St. Paul.

This paper is based on a study by Stanley Consultants, Inc., conducted for the Metropolitan Council to evaluate the potential for land application systems for treatment of municipal wastewater. The Administrator of the U.S. Environmental Protection Agency has recently emphasized utilization of land application systems (1). Land application systems can achieve a high degree of wastewater treatment and are most likely to be cost-effective where stream water quality standards require tertiary treatment of biologically treated effluents. If land application systems appear cost-effective for certain areas they will be investigated in more detail during Facility Planning studies conducted under Section 201 of the Federal Water Pollution Control Act.

Objectives

The purpose of this study was to develop the tools necessary to evaluate the potential for land application systems in the study area. Specific work tasks required to complete the evaluation include:

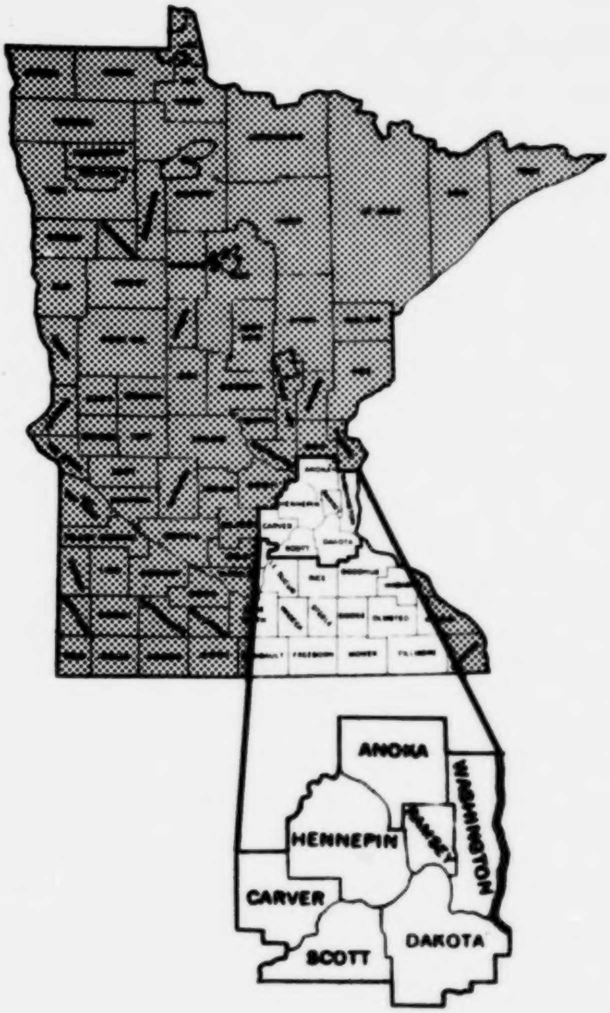
1. Development of criteria for the preliminary design and costing of land application systems in the study area.
2. Development of construction and operation and maintenance costs for land application systems under analysis. In addition, costs for transmission of wastewater to the land application site are developed.
3. Assessment of climatological, land use, soils, and topographic data pertinent to land treatment of wastewater in the study area.
4. Identification of potential sites for land application in the study area.

Study Area

The study area consists of a 7-county area surrounding the cities of Minneapolis and St. Paul (Figure 1). The 1976 study area population is estimated to be about 1,993,000, which is the 14th largest urban area in the USA. Minnesota is one of the most northern states with Minneapolis-St. Paul located along latitude 45°. Winters are rather severe with an average of 160 days where the minimum daily temperature is equal to or below 32°F (0°C). Average yearly snowfall is about 42 in. (107 cm) and the growing season is about 120 days.

The majority of the wastewater transmission and treatment systems in the study area are owned and operated by the Metropolitan Waste Control Commission (MWCC). The MWCC is the designated 201 Agency for preparation of more detailed facility plans. The existing system consists of 20 treatment plants, 400 miles (644 km) of interceptor sewers and 40 lift stations. The system served 100 communities and approximately 1.8 million persons in 1976. Wastewater treatment facilities range in size from the 200 mgd (757,000 m^3/d) Metropolitan Plant to facilities ≤ 1 mgd ($\leq 3,785$ m^3/d).

The study area has a large number of lakes and three major river systems which receive most of the wastewater treatment plant discharges. These rivers are the Mississippi, Minnesota, and St. Croix. Although all of the final effluent standards for the wastewater treatment facilities are not yet fully established, certain stream segments are water quality limited and some effluent standards may be more stringent than secondary treatment (30



mg/l BOD_5 and 30 mg/l TSS). Phosphorus removal to 1 mg/l is required for facilities discharging to the St. Croix River. It is the smaller communities, outlying Minneapolis-St. Paul, where land application of wastewater appears to have the most promise for being cost-effective.

Land Treatment Processes

General

There are three principal processes used for land application of municipal wastewaters which were considered for use in this study area. These three processes are:

1. Slow rate.
2. Rapid infiltration.
3. Overland flow.

A fourth lesser-used process in which wastewater is applied to peatland for renovation was also briefly investigated since there are many peat deposits in the study area. Although peat soils have been used to treat secondary effluent at several campgrounds in Minnesota with good results (2, 3), the use of peat for larger scale systems appears to have limitations because of extensive and expensive site preparation requirements. This, coupled with no actual operating experience with large scale land treatment systems using peat soils, preclude further consideration of these sites for land treatment in the study area.

Slow Rate

In the slow rate land treatment process, sometimes referred to as irrigation, wastewater is treated as it flows through the soil system. Vegetative cover removes nitrogen from the wastewater while helping to keep the infiltration capacity of the soil at an acceptable rate to receive the volume of applied wastewater. Wastewater is applied using either surface or sprinkler application techniques. The wastewater applied either evapotranspirates or percolates to the groundwater. The hydraulic loading rate for this process is limited by either the infiltration capacity of the soil or the nitrogen removal capacity of the soil-vegetation system. This treatment system provides the highest level of treatment and the most reliability of the three treatment processes under consideration.

Rapid Infiltration

The rapid infiltration treatment process, sometimes referred to as infiltration-percolation, applies large volumes of wastewater to the land for treatment. The soils must be rapidly permeable, and vegetation is not usually required. The applied wastewater percolates through the soil and reaches the groundwater unless recovered by the use of underdrains or recovery wells. The wastewater may be applied by spreading in basins or by sprinkling.

This method of land treatment uses the filtering and straining action of the soil as well as biological activity to remove suspended solids, BOD, and fecal coliform. Although this method of treatment results in a highly nitrified effluent, overall nitrogen removal is generally poor. Phosphorus removal varies with the type of soil and travel distance of the wastewater through the soil, but expected quality after treatment is still in the 1 to 5 mg/l range (4).

Overland Flow

Wastewater is applied to the upper portions of sloping, relatively impermeable soil in the overland flow process. The wastewater is treated as it flows down the slope, and a large portion of the water applied appears as run-off at the bottom of the slope. This method has not been widely used for municipal wastewater treatment and does not provide as great a degree of removal of BOD, suspended solids, and fecal coliform as the two previously discussed methods of treatment.

Land Treatment Process Comparisons

Design features for slow rate, rapid infiltration, and overland flow land treatment processes are compared in Table 1. Table 2 compares site characteristics, while Table 3 shows the expected quality of treated wastewater from the various land treatment processes.

Table 1

Comparison of General Design Features for Land Treatment Processes

		Processes	
Feature	**Slow Rate**	**Rapid Infiltration**	**Overland Flow**
Application techniques	Sprinkler or surface^1	Usually surface	Sprinkler or surface
Annual application rate, ft^6	2 to 20	20 to 560	10 to 70
Field area required, acres^{2,7}	56 to 560	2 to 56	16 to 110
Typical weekly application rate, in.^{8}	0.5 to 4	4 to 120	2.5 to 6^3 6 to 16^4
Minimum pre-application treatment provided in United States	Primary sedimentation^5	Primary sedimentation	Screening and grit removal
Disposition of applied wastewater	EVapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Need for vegetation	Required	Optional	Required

Notes: [1] Includes ridge-and-furrow and border strip.

[2] Field area in acres (ha) not including buffer area, roads, or ditches for 1 mgd flow.

[3] Range for application of screened wastewater.

[4] Range for application of lagoon and secondary effluent.

[5] Depends on the use of the effluent and the type of crop.

[6] 1 ft = 0.3048 m

[7] 1 acre = 0.40469 ha

[8] 1 in. = 2.54 cm

Table 2

Comparison of General Site Characteristics for Land Treatment Processes

		Processes	
Characteristics	**Slow Rate**	**Rapid Infiltration**	**Overland Flow**
Slope	Less than 20% on cultivated land; less than 40% on non-cultivated land	Not critical; excessive slopes require much earthwork	Finish slopes 2 to 8%
Soil Premeability	Moderately slow to moderately rapid	Rapid (sands, loamy sands)	Slow (clays, silts, and soils with impermeable barriers)
Depth to	2 to 3 ft (minimum) (0.61 to 0.91 m)	10 ft (3.05 m) (lesser depths are acceptable where underdrainage is provided	Not critical
Climatic Restrictions	Storage often needed for cold weather and precipitation	None (possibly modify operation in cold weather)	Storage often needed for cold weather

Source: Reference (4)

Land Treatment Design Criteria

General

There are a number of local factors which must be considered in selection of potential land treatment systems and development of design criteria for the systems selected. This section presents the local factors that must be taken into consideration, identifies land treatment processes which appear to have potential for use in the study area, and evaluates design criteria for the selected land treatment systems.

Local Factors

Raw Wastewater Characteristics — Typical raw wastewater characteristics for municipal wastewater in the study area were developed in a previous study (5). These characteristics are as follows:

BOD_5 (5-day biochemical oxygen demand)	225 mg/l
TSS (total suspended solids)	250 mg/l
NH_3-N (ammonia nitrogen)	20 mg/l
Organic-N (organic nitrogen)	11 mg/l
P (phosphorus)	9 mg/l

It has been assumed for this study that concentrations of heavy metals in the predominantly municipal wastewater are not significant enough to affect application of wastewater at the annual hydraulic loading rates discussed in this report. Industries contributing wastewater to the municipal wastewater collection systems would be required to pretreat their discharges to the degree where heavy metal interference with land application would not occur.

Table 3

Expected Quality of Treated Water from Land Treatment Processes
mg/l

Constituent	Slow Rate[1] Average	Slow Rate[1] Maximum	Rapid Infiltration[2] Average	Rapid Infiltration[2] Maximum	Overland Flow[3] Average	Overland Flow[3] Maximum
BOD	<2	<5	2	<5	10	<15
Suspended Solids	<1	<5	5	<10	20	
Ammonia Nitrogen as N	<0.5	<2	0.5	<2	0.8	<2
Total Nitrogen as N	3	<8	10	<20	3	<5
Total Phosphorus as P	<0.1	<0.3	1	<5	4	<6

Note: [1] Percolation of primary or secondary effluent through 5 ft (1.52 m) of soil.
[2] Percolation of primary or secondary effluent through 15 ft (4.57 m) of soil.
[3] Runoff of comminuted municipal wastewater over about 150 ft (45.7 m) of slope.

Source: Reference (4).

This study also assumes that raw wastewater will be transmitted to the land application site where the necessary pretreatment takes place. At a minimum, primary clarification is required prior to land application to prevent plugging of distribution systems and soil clogging. In certain cases secondary treatment may be desirable. This study does not address the cost-effectiveness of utilizing existing wastewater treatment facilities for pretreatment prior to transmitting the wastewater to the land application site.

Climate — Climatic information is based primarily on data collected at the U.S. Weather Bureau station at the Minneapolis-St. Paul airport. Climatological data are used in the calculation of water balances, wastewater application rates and periods, and storage requirements. Table 4 presents climatic data for the study area.

Soils — Soil characteristics are important factors to be considered when evaluating land treatment alternatives. Soil properties affect the choice of process and application method, thereby greatly influencing the land area requirements. Soil characteristics evaluated during this study included soil depth, texture (root zone and substratum), permeability, and slopes. Evaluation of soil characteristics was conducted based on general conditions occurring over a broad area versus specific site evaluations. No actual field testing or analysis of site specific field data was conducted.

Sources of information utilized in evaluating soil characteristics included USGS quadrangle maps; publications and maps of the Soil Conservation Service, USDA; and the Agricultural Extension Service and Agricultural Experiment Station, at the University of Minnesota (11, 12, 13, 14, 15, 16, 17).

Groundwater Table — The depth of the groundwater table determines the depth of unsaturated soil in which renovation will occur and, therefore, the degree of treatment which will take place in the soil. Groundwater depth was determined from USGS quadrangle maps in combination with an areawide water table elevation contour map (18) and references listed in the preceding soils subsection of this report.

A portion of the wastewater applied to the soil by land treatment systems will eventually percolate to the groundwater if not intercepted by underdrains or recovery wells. The treated wastewater must not raise groundwater constituent concentrations to levels exceeding EPA Interim Primary Drinking Water Standards if the groundwater is being used or could be used as a source of drinking water supply.

Land Use — Land use information was based on USGS quadrangle maps, exclusion maps compiled by the Minnesota Land Management Information Center computer system, and projected growth of urban areas through the year 2000 from the Metropolitan Council.

Land Treatment Process Selection

Based on analysis of local factors discussed previously and data from numerous references (4, 19, 20, 21, 22, 23, 24, 25), slow rate and rapid infiltration systems are considered to be feasible land treatment systems for use in the study area. Overland flow systems were ruled out due to their lower level of treatment and the lack of extensive areas having soils with the proper permeability and slope relationships for use with this treatment process.

Slow rate systems proposed in this study are based on the treatment of wastewater, rather than the maximization of crop yields as is sometimes considered for cropland irrigation systems. This choice was made because application of wastewater solely to supplement natural rainfall consumed by crops requires much larger land areas since the wastewater cannot be applied at as high a rate or for as long a period as when grasses are grown. Grasses take up more nutrients than do crops such as corn, allowing higher nutrient and hydraulic loadings. Although there is a larger economic return from corn or other crops as opposed to grasses, slow rate systems which maximize wastewater treatment are considered

Table 4

Climatic Data for the Twin Cities Metropolitan Area

	Temperature, °F (°C)		Mean Number of Days		Precipitation (Pr)					Prevailing Direction and Mean Speed of Wind	
Month	Mean	Mean Daily Minimum	Maximum 32° (0°) 0 Below	Minimum 32° (0°) & Below	Average Annual (in.)[4]	Wettest[1] Year in 10 (in.)[4]	Mean Number of[2] Days with Pr 0.5 In.[4]	(ET)[3] Evapotranspiration (in.)[4]	Monthly Net Water Excess Pr-ET (in.)[4]	Direction Wind From (miles/hr)[5]	Mean Speed
January	12	3	24	31	0.73	1.05	1	—	10.5	NW	11
February	16	5	19	28	0.84	1.12	2	—	1.12	NW	11
March	28	18	11	27	1.68	1.96	4	—	1.96	NW	12
April	45	33	<0.5	12	2.04	2.71	4	1.32	1.39	NW	13
May	58	46	0	2	3.37	4.44	4	3.32	1.12	SE	12
June	67	56	0	0	3.94	5.27	7	5.08	0.19	SE	11
July	73	61	0	0	3.69	4.57	4	5.87	-1.30	S	9
August	71	59	0	0	3.05	4.12	4	4.87	-0.75	SE	9
September	61	49	0	1	2.73	3.79	6	3.15	0.65	S	10
October	48	37	0	7	1.78	2.65	4	1.69	0.96	SE	11
November	31	21	6	22	1.20	1.73	2	—	1.73	NW	12
December	18	9	22	30	0.89	1.20	1	—	1.20	NW	11
			82	160	25.94	34.61	43	25.30	9.31		

Notes: [1] The design annual precipitation was determined on the basis of a frequency analysis for the wettest year in 10 utilizing data from the period 1934-1973. The monthly totals are based on the average percentage of the annual precipitation that occurs in each month.

[2] Based on eleven (11) years of records, 1964-1974.

[3] Based on Thornthwaite method for period 1931-1969.

[4] 1 in. = 2.54 cm.

[5] 1 mile/hr = 1.609 km/h

Source: References (6, 7, 8, 9, 10, 26) and U.S. Weather Bureau Data, Minneapolis-St. Paul, Minnesota.

more practical for the study area. The length of time over which wastewater can be applied during the year is rather short, even with grasses, due to local climatic conditions.

Slow rate systems will provide sufficient treatment to the wastewater so that the percolate can discharge to the groundwater without causing adverse effects. Only one hydraulic loading rate and set of application criteria have been developed for the slow rate system, since most soils in the study area which are best suited for slow rate land treatment have a similar permeability range.

There are soils in the study area which have permeabilities allowing use of rapid infiltration systems. Rapid infiltration systems require much less land area than do slow rate systems, but the degree of treatment provided by this method is not as great as that for the slow rate process. The degree of effluent quality required prior to release of percolate to the groundwater is based on current or potential aquifer usage. If the aquifer is presently or could potentially be used as a source of drinking water supply, groundwater quality must meet U.S. EPA Interim Primary Drinking Water Standards. Within the study area, there are bedrock aquifers 100 to 300 ft (30.8 to 92.5 m) below the surface which are used primarily for drinking water. In general, these aquifers are sealed from overlying aquifers, but there is no way of assuring that water percolating downward will not reach the deep aquifers. Therefore, rapid infiltration systems would require collection of the percolate for discharge to surface waters so that nitrate nitrogen standards (<10 mg/l) would not be exceeded in the groundwater.

Rapid infiltration systems may not remove phosphorus to a level <1 mg/l, so that discharge of the recovered percolate would be limited to streams or rivers which do not have a phosphorus limitation. This means that rapid infiltration system effluent could not be discharged to the St. Croix River or area lakes. Since two distinct ranges of soil permeabilities are availble for use with rapid infiltration systems in the study area, two different design criteria have been developed for rapid infiltration treatment. No special crops are grown on the infiltration basins where the wastewater is applied, although it is anticipated that a grass cover would be maintained.

Design Criteria Usage

The design criteria proposed in this study are based on climatological, groundwater table, and land use data in combination with general soil physical, hydraulic, and chemical properties. The data base was determined from references listed in the preceding "Local Factors" subsection. This information was used in conjunction with listed references and discussions with researchers at the USDA Agricultural Research Service concerning their findings at a wastewater land application test site in Apple Valley, Minnesota, to establish the proposed land treatment design criteria. While the design criteria and cost information developed for this study are adequate for preliminary planning purposes, it must be emphasized that detailed and site specific analyses must be conducted to assure that a given site is suitable for land treatment. Field testing of soil properties, analysis of groundwater quality and location, and evaluation of site topography and vegetation are required to establish final land treatment process design criteria.

Hydraulic Loading

Slow Rate — The allowable hydraulic loading of wastewater for slow rate systems is affected by soil permeability, climate, and crop selection. The permeability of soils in the study area which are considered most suitable for slow rate land treatment fall within the 0.6 to 2 in./hr (1.5 to 5.1 cm/h) category, which is described by the SCS as "moderate" permeability. Figure 2 presents application rates which are suitable for this range of soil permeability.

Using water balance and climatic data and Figure 2, an annual wastewater loading rate of 55 in./yr (140 cm/y) has been selected for slow rate systems in the study area. This loading rate is based on the use of reed canarygrass as a cover crop and application over a 26-week period per year. Application of 55 in. (140 cm) over a 26-week period results in an average weekly application rate of 2.11 in (5.36 cm). Since mean annual evapotranspiration and precipitation in the study area nearly balance, the 2.11 in./wk (5.36 cm/wk) average application rate falls near the middle of application rates shown on Figure 2 used in practice for moderate permeability soils.

Table 5 shows weekly application rates proposed for use throughout the application period. Wastewater would generally be applied at a rate of approximately 0.25 in./hr (0.64 cm/h) for a period of 4 to 16 hr one day per week, with no wastewater application the following 6 days. If soil conditions or management practices dictated, the application time could be halved and twice weekly application could be utilized.

Reed canarygrass was selected as the forage crop for slow rate systems due to its good water tolerance, its maintenance of high surface infiltration rates, and its high nutrient uptake (23). It is assumed that the reed canarygrass would be harvested three times per year in order to maximize nutrient uptake and crop quality. The harvests would occur approximately the first week in June, the last week in July, and the latter part of September. The Minnesota Department of Agriculture estimates that the value of the harvested reed canarygrass is about $50 per ton as a forage crop for domestic animals and that the annual yield would be about 2.5 tons/acre (918 kg/ha) for a total crop value of about $125 per acre ($51/ha).

Reed canarygrass generally requires a year to become established. Therefore, another grass such as orchard or ryegrass should be planted simultaneously with the reed canarygrass to provide a crop the first year of wastewater application.

Table 5

Proposed Application Periods and Rate[1] for Slow Rate Land Treatment Systems

Approximate Dates	Application Amount (in./acre/wk)[3]	Total During Period (in.)[4]
23 April-14 May	1	3
15 May-22 June[2]	2	8
23 June-September[2]	4	36
2 September-30 September[2]	2	6
1 October-15 October	1	2
		55

Notes: [1] Application amounts and dates shown should be modified in actual operation to reflect current temperature, moisture, crop requirements, and overall management objectives.
[2] Indicates no wastewater application during one week of this period for purposes of crop harvest.
[3] 1 in./acre/wk = 6.276 cm/ha/wk.
[4] 1 in. = *.54 cm*

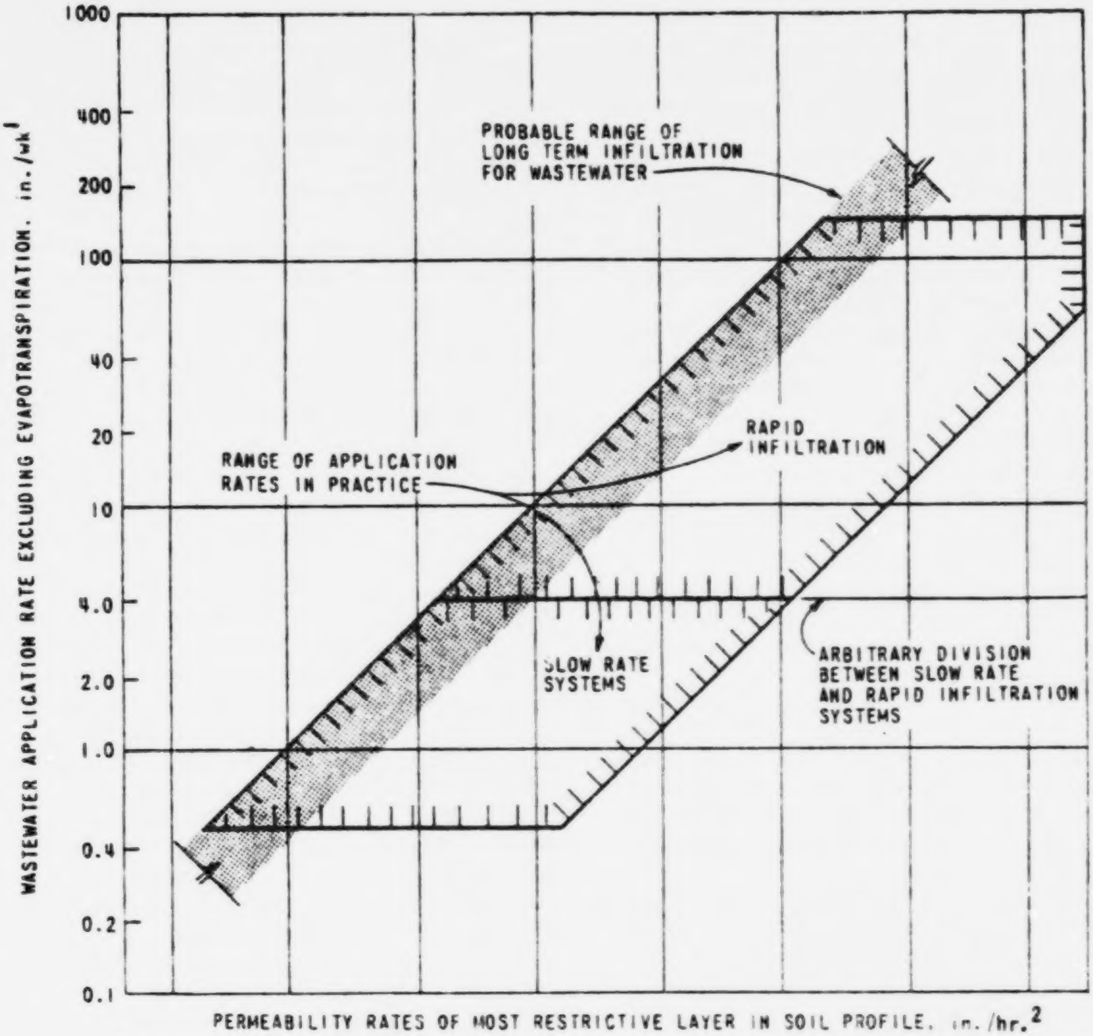
Source: Stanley Consultants, Inc.

Hydraulic loading rates proposed in this study exceed guidelines established by the Minnesota Pollution Control Agency. PCA regulations allow a spray irrigation season of only 18 weeks and an overall maximum hydraulic loading of 52 in./acre/yr (326 cm/ha/y). For purposes of determining land requirements, the maximum application allowable by the PCA regulations is 36 in./acre/yr (226 cm/ha/y).

Rapid Infiltration — As mentioned previously, two different rapid infiltration application rates are proposed. The first application rate is based on applying 15 in./wk (38.1 cm/wk) for 8 months of each year. This amounts to 520 in./yr (1,321 cm/y), or 43 ft/yr)13.1 m/y). This hydraulic loading rate would be used with those soils having permeabilities in the range of 4 to 6 in./hr (10.2 to 15.2 cm/h). The second hydraulic loading rate is associated with sites where permeabilities of 12 to 14 in./hr (30.5 to 35.6 cm/h) are available. For this high permeability, an application rate of 28 in./wk (71.1 cm/wk) could be used on an 8-month/yr basis, resulting in a total application of 960 in./yr (2,438 cm/y), or 80 ft/y (24.4 m/y). The application cycle for these systems would be approximately 2 weeks, with wastewater applied to an infiltration basin for 1 to 4 days and a resting period of 10 to 14 days before additional application of wastewater would be allowed.

Land Area Requirements

Land is required for preapplication treatment lagoons, storage lagoons, wastewater application areas, buffer zones for spray application systems, buildings, service roads, and other miscellaneous items. The area actually receiving direct wastewater application is termed the "wetted area" and is related directly to the total annual wastewater application.



PERMEABILITY*, SOIL CONSERVATION SERVICE DESCRIPTIVE TERMS						
VERY SLOW	SLOW	MODERATELY SLOW	MODERATE	MODERATELY RAPID	RAPID	VERY RAPID
< 0.06	0.06-0.20	0.20-0.60	0.60-2.0	2.0-6.0	6.0-20.0	> 20.0

* MEASURED WITH CLEAR WATER

[1] 1 in./wk. = 2.54 cm./wk.

[2] 1 in./hr. = 2.54 cm./hr.

SOURCE: REFERENCE (4)

Figure 2
Soil Permeability Versus Ranges of Application Rates for Slow Rate and Rapid Infiltration Treatment

Slow Rate — This land treatment system requires 244 "wet acres" per mgd (0.026 ha/m^3/d). Total land area for this system would be approximately 350 acres/mgd (0.037 ha/m^3/d) (4). These land area requirements may be considered sufficiently accurate for planning purposes, but there may be additional quantities of land for certain treatment sites which are not readily usable for wastewater application. Sites containing tracts of small hilltops and valleys, steep slopes, or unsuitable soils will add to the overall land area required at a given site per mgd (m^3/d) of wastewater treated.

Buffer zones of approximately 200 ft (61 m) in width will be provided around the perimeter of all spray application areas. These buffer areas increase the overall land requirements for the slow rate system, but are considered necessary to help prevent carryover of aerosols resulting from the spray application system.

Rapid Infiltration — The lower rate rapid infiltration system would require a "wet acreage" of 26 acres per mgd (0.0028 ha/m^3/d), while the higher rate rapid infiltration system would require only 14 "wet acres" per mgd (0.0015 ha/m^3/d). Total land area for the rapid infiltration systems is based on land for preapplication treatment, storage, wastewater application areas, buildings, roads, etc, but does not include buffer zones, as were required for spray application systems in the slow rate land treatment process. Total land area for the first rapid infiltration system is approximately 52 acres/mgd (0.0056 ha/m^3/d), while total land area for the second rapid infiltration system is approximately 35 acres/mgd (0.0037 ha/m^3/d).

Land Acquisition — The outright purchase of land for land treatment is usually considered to be the best land acquisition method (4). Direct purchase of the land allows more control over application site management and allows the controlling agency a greater degree of freedom in terms of long-range planning. Financing of land to be used in the treatment process is generally eligible for PL 92—500 construction grant funding. Long-term leases may also be eligible for construction grant funding if they can be shown to be more cost-effective than outright purchase of the land.

Constituent Loadings

Nitrogen — The nitrogen balance for slow rate systems is a function of the nitrogen applied, the nitrogen removed by crop uptake, nitrogen lost by denitrification and volatilization, and nitrogen lost to the groundwater. The percolate to the groundwater must contain ≤ 10 mg/l of nitrate nitrogen in order to meet requirements for discharge to groundwaters that are presently or could potentially be used as a source of drinking water supply.

Reed canarygrass, proposed for use on slow rate land treatment application areas, can be assumed to uptake approximately 300 lb/acre/yr (336 kg/ha/y) of nitrogen with the rate of application of nitrogen to the soil and the number of harvests. Three harvests have been assumed over the application season in the study area.

Denitrification in slow rate systems can be assumed to range from approximtely 15 to 25 percent of the applied nitrogen (4). For purposes of the nitrogen balance computations, denitrification was estimated at 15 percent of the applied nitrogen, since denitrification rates are lowest for uniform soils with moderate to rapid permeabilities (24).

Table 6 presents the nitrogen balance calculations for the slow rate system proposed for use in the study area. Nitrate nitrogen concentrations are very low, thereby assuring that 55 in./yr (140 cm/y) of wastewater could be applied without exceeding nitrogen limitations. The nitrogen balance indicates that 293 lb (113 kg) of nitrogen would be applied per year with application of 55 in. (140 cm) of wastewater containing 23.5 mg/l of nitrogen which is the estimated total nitrogen in the effluent from the pretreatment facilities (total nitrogen in the typical raw wastewater for the study area is 31 mg/l). To assure maximum crop yield, the nitrogen balance indicates that it might be necessary to apply a small amount of supplemental nitrogen during the early part of the growing season. It is doubtful that additional fertilizer would be added, but rather a slightly lower crop yield would probably be accepted. It becomes apparent that if soil tests at a specific site indicate a higher annual application rate could be utilized from a hydraulic standpoint, nitrogen limitations will not preclude use of application rates in excess of 55 in./yr (140 cm/y). Experimental work conducted by USDA Agricultural Research Service and University of Minnesota researchers at an Apple Valley land application test site indicated that application of up to 95 in./yr (241 cm/y) of secondary wastewater effluent /365 lb N/acre (409 kg N/ha)/ on reed canarygrass did not result in unacceptable nitrate concentrations in the percolate (23). This application rate would be impossible to achieve on a full-scale operation due to problems with groundwater mounding and maintenance of a farming operation, but illustrates that nitrogen limitations should not be a problem with the use of three harvests of reed canarygrass and practical wastewater application rates.

Rapid infiltration systems rely on nitrification, denitrification, and ammonium absorption for nitrogen removal. Rapid infiltration systems do not utilize a crop in the application area, so nitrogen removal by crop uptake is minor. Generally, rapid infiltration systems will produce a highly nitrified effluent, but total nitrogen removal is usually not great enough to allow percolation of the effluent to the groundwater in cases where the groundwater is or can be used for drinking water supply. Therefore, recovery of the applied wastewater by use of underdrains or recovery wells is proposed in this study, with discharge of the percolate to surface waters.

Phosphorus — Most of the phosphorus applied to the soil in the slow rate system is utilized by crops or retained in the soil by means of adsorption and chemical precipitation reactions. Harvest of reed canarygrass in the study area could be assumed to remove approximately 40 lb/acre/yr (45 kg/ha/y) of phosphorus. Most of

Table 6

Slow Rate System
Monthly Nitrogen Balance

Month	Pr-ET,[1] Net Monthly Excess Water (in.)[6]	I.w. Applied Wastewater (in.)[6]	I.n.[2] Wastewater Nitrogen Loading (lb/acre)[7]	D.[3] Denitrification (lb/acre)[7]	Maximum[4] Potential Crop Nitrogen Update (lb/acre)[7]	Wp. Percolate Water (lb/acre)[7](in.)[6]	Pn. Percolate Nitrogen (lb/acre)[7]	Cp. Percolate Nitrogen Concentration (mg/l)
January	1.05	0	0	0	0	—	—	—
February	1.12	0	0	0	0	—	—	—
March	1.96	0	0	0	0	—	—	—
April	1.39	1.0	5.33	0.80	15.65	2.39	0[5]	—
May	1.12	6.0	32.01	4.80	39.37	7.12	0[5]	—
June	0.19	8.0	42.68	6.40	60.24	8.19	0[5]	—
July	-1.30	16.0	85.35	12.80	69.60	14.7	2.95	0.88
August	-0.75	16.0	85.35	12.80	57.75	15.25	14.80	4.28
September	0.64	6.0	32.01	4.80	37.35	6.64	0[5]	—
October	0.96	2.0	10.67	1.60	20.04	2.96	0[5]	—
November	1.73	0	0	0	0	—	—	—
December	1.20	0	0	0	0	—	—	—
		55.0	293.40	44.0	300.0			

Notes: [1] From Table 5.
[2] Assumes applied nitrogen concentration of 23.5 mg/l.
[3] Assumes denitrification equals 15 percent of nitrogen applied.
[4] Assumed maximum reed canarygrass uptake if adequate nitrogen is available. Monthly uptake is assumed to correspond with ratio of monthly evapotranspiration over annual evapotranspiration.
[5] If the percolate contains nitrogen, the quantity should be very low.
[6] 1 in. = 2.54 cm
[7] 1 lb/acre = 1.12 Kg/ha

Source: Reference (4) and Stanley Consultants, Inc.

the remainder of the applied phosphorus will remain in the soil and will result in the phosphorus adsorption capacity being reached after a number of years. The actual life of an application site can be computed once the phosphorus adsorption capacity of the soil is analyzed by means of laboratory testing of field samples.

Rapid infiltration systems do not utilize a crop, so phosphorus removal by crop uptake is minor. Rapid infiltration systems also use coarser textured, more permeable soils than do the slow rate systems, resulting in less retention of phosphorus in the soil matrix than soils associated with slow rate systems. Actual removal of phosphorus by rapid infiltration soils depends on soil pH and the levels of free iron oxides, calcium, and aluminum. It has been assumed in this analysis that rapid infiltration systems will not reliably produce an effluent with a phosphorus concentration ≤ 1 mg/l. Therefore, percolate from rapid infiltration treatment cannot be discharged to phosphorus limited rivers and lakes in the study area.

Biochemical Oxygen Demand and Suspended Solids — Slow rate and rapid infiltration systems both provide a high percentage of BOD and SS removal. Table 3 shows expected quality for these parameters following land treatment by these two processes.

Preapplication Treatment

Slow Rate Systems — Although reduction of organic and suspended solids material in the wastewater is not necessry from a soil treatment standpoint, preapplication treatment is necessary to reduce these parameters so that a reliable distribution system may be maintained. It is also advantageous to reduce organic and suspended solids so that storage of wastewater does not create nuisance conditions, soil clogging is reduced, and health risks due to spray of aerosols are minimized. For these reasons, preapplication systems proposed for slow rate systems consisting of aerated lagoons providing 6 days detention. Treatment in the aerated lagoons and further treatment in the storage lagoons, coupled with use of 200-ft (61 m) buffer zones and fencing of the entire site, should allow spray application to be conducted without prior disinfection. Aerated lagoons are assumed to have an average working depth of approximately 15 feet (4.6 m), and will use surface floating aerators to meet the oxygen demand of the wastewater.

Rapid Infiltration — These land treatment systems will also require preapplication treatment consisting of aerating lagoons providing 6 days detention. The aerated lagoons remove settleable solids that could cause clog-

Table 7

Slow Rate System Storage Determination (In. of Water)[1]

Month	Precipitation[2]	Effluent Applied	Evapotranspiration[3]	Required Percolation (1 + 2 − 3)	Effluent Available	Change in Storage (5 − 2)	Total[4] Storage
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
January	1.05	0	—	1.05	4.66	4.66	16.49
February	1.12	0	—	1.12	4.36	4.36	20.85
March	1.96	0	—	1.96	4.66	4.66	25.51
April	2.71	1.0	1.32	2.39	4.51	3.51	29.02
May	4.44	6.0	3.32	7.12	4.66	-1.34	27.68
June	5.27	8.0	5.08	8.19	4.51	-3.49	24.19
July	4.57	16.0	5.87	14.70	4.66	-11.34	12.85
August	4.12	16.0	4.87	15.25	4.66	-11.34	1.51
September	3.79	6.0	3.15	6.64	4.51	-1.49	0.02
October	2.65	2.0	1.69	2.96	4.66	2.66	2.66
November	1.73	0	—	1.73	4.51	4.51	7.17
December	1.20	0	—	1.20	4.66	4.66	11.83
	34.61	55.0	25.3	57.25[5]	55.0		

Note: [1] 1 in. = 2.54 cm.
[2] From Table 5, based on wettest year in 10.
[3] From Table 5.
[4] Storage reservoir empty at the end of September.
[5] Does not include precipitation in November, December, January, February, or March.

Source: Stanley Consultants, Inc.

ging of the soil at the high loading rates applied in rapid infiltration systems. In addition, the aerated lagoons will help to minimize nuisance conditions associated with storage of the wastewater during winter periods. Design criteria similar to those discussed previously for slow rate systems will be utilized.

Storage Requirements

Slow Rate — Slow rate land treatment systems require storage to hold the wastewater flow which cannot be applied during periods of inclement weather. Based on the application schedule in Table 5, the maximum storage volume for the slow rate system can be calculated. Table 7 illustrates the quantity of storage required, based on a mass balance between wastewater applied and wastewater available. A total quantity of 29.02 in. (73.7 cm) of wastewater must be stored, translating to a storage volume of 592.5 acre-ft/mgd (193 $m^3/m^3/d$). This volume represents the equivalent of 193 days flow. The maximum storage occurs in April when the application season is just starting. Applying a safety factor for storage of an additional 2 weeks flow in the case of an exceptionally late or wet spring, or incomplete emptying of the lagoon at the end of the previous application season, results in a proposed storage lagoon requirement of 207 days flow, or 635.5 acre-ft/mgd (207 $m^3/m^3/d$).

Rapid Infiltration — Although several references (4, 19, 20) indicate that rapid infiltration land treatment systems can often be operated on a continuous year-round basis, review of literature on systems located in severe northern climates similar to that experienced in the study area indicates that these systems apply warm wastewater to infiltration basins located immediately adjacent and following mechanical treatment facilities. Therefore, the temperature of the applied wastewater is still high enough to allow the wastewater to percolate through the permeable soil without causing freezing problems. Sometimes ice forms on the infiltration basins, but the warm wastewater is reported to float the ice and infiltrate through the soil underneath. In systems proposed in the study area, however, the wastewater will be transmitted from the community collection systems through transmission lines to the aerated lagoon where it is held for 6 days. Although the aerators should keep the wastewater mixed enough to prevent total freezing, the temperture of the applied wastewater is going to be near 32°F (0°C) in extremely cold weather. Application of this near freezing wastewater to the infiltration basin would probably result in freezing of the surface and water in the soil pores, resulting in extreme operational difficulties. Therefore, it has been assumed that those periods of extreme climatic conditions (December, January, February, and March) will not be conducive to continuous wastewater application. A storage lagoon of adequate size to hold the volume of wastewater generated during this period is proposed. The volume of the lagoon would be sufficient to hold 120 days flow or require 368.3 acre-ft/mgd (120 $m^3/m^3/d$) of wastewater to be treated.

Design Criteria — Storage lagoons for both slow rate and rapid infiltration systems are assumed to have an operating depth of 12 feet (3.7 m). A lining of asphaltic material is proposed for the entire inside area of the reservoir to prevent percolation of the wastewater through the bottom of the storage lagoon into the groundwater prior to land application. Lagoons also will require riprap on the inside slope of the dikes in order to prevent erosion of the embankment by wind-created wave action. a 3-foot (0.91 m) freeboard is assumed above high water level. Large storage reservoirs are assumed to be divided into multiple cells.

Application Systems

Slow Rate — Wastewater may be applied using either surface or sprinkler techniques for slow rate systems. Surface application, however, requires relatively flat land in order to minimize the amount of earthwork necessary to prepare the application sites for wastewater distribution. Sprinkler distribution is less dependent on slope considerations. Sprinkler distribution may be accomplished by either fixed, solid set permanent sprinklers or hand or mechanically moved sprinkling systems. Due to the somewhat rolling terrain found in the study area, and savings on labor over hand-moved systems, center pivot mechanically moved irrigation systems have been proposed for use for most application sites. Center pivot systems will work well in large open areas, but solid set systems may be used to advantage in smaller, odd shaped fields, even though solid set systems hamper the planting and harvesting of crops in the field area. Costs presented later in this report are based on the use of center pivot spray systems.

Rapid Infiltration — Application of wastewater for rapid infiltration systems usually consists of surface flooding of infiltration basins which are constructed on near-level ground and surrounded by dikes. The basins should generally be flat in order to allow uniform distribution of the applied wastewater over the surface. In order to minimize the amount of earthwork required in construction, infiltration basins are best located in relatively flat areas. It is possible to contract basins by terracing into the side slopes of steeper areas, but application of wastewater in an upper basin may adversely effect application in lower basins further down the slope. Multiple basins are generally required to provide flexibility in operation of the system and to insure adequate drying time between application periods. Vegetation is not required on the surface of the basin, but a grass or other vegetative cover generally helps to maintain the infiltration rate of the soil surface.

Subsurface Drainage

Slow Rate — The ground water table must be controlled at level that will provide sufficient soil detention time and travel distance to insure that the required degree of treatment is achieved. For slow rate systems, it is necessary to maintain a groundwater table approximately 5 ft (1.52 m) below the ground surface. The addition of percolated wastewater will tend to raise the natural ground water level. Areas of the application site which already have or develop on a high groundwater table will require artificial drainage to maintain the groundwater table at the desired depth.

Application sites selected in the study area will generally not require subsurface drainage over the entire area, but natural drainageways and lower portions of natural sloping areas may require underdrains to maintain proper groundwater depths. Treated wastewater collected in the underdrains from these areas will be discharged to adjacent surface streams.

Costs for slow rate systems do not include the cost of underdrains, since this is a site specific requirement. For planning purposes, however, construction of underdrain systems can be assumed to add approximately $1,000 per underdrain acre ($405/ha /based on spacing of 250 ft (76 m)/, and have annual operating costs of approximately $150/acre/yr ($61/ha/y) for an underdrained area of 25 acres (10.1 ha) to approximately $40/acre/yr ($16/ha/yr) for an underdrained area of 2,500 acres (1,012 ha).

Rapid Infiltration — Rapid infiltration systems require that soils be adequately drained to maintain desired infiltration rates. Subsurface drainage will insure that the soil is unsaturated and has adequate resting periods between applications for renovation. In addition, it is necessary to collect the percolate for discharge to surface waters, since groundwater quality standards would be violated by rapid infiltration system effluent.

Percolate can be collected by either underdrain systems or recovery wells. It is necessary that underdrains be located below the water table in sandy soils so that they will collect the percolating water. In those cases where the water table is deep and the soils where the water table is located are permeable enough, the percolate would be recovered by pumped withdrawal rather than underdrains.

It has been assumed that pumped withdrawal will be the most suitable method of recovering the rapid infiltration land treatment percolate in the study area. Costs for rapid infiltration systems include the cost of recovery wells for percolate collection and discharge. In those cases where underdrains may be suitable, a spacing of 400 ft (122 m) at 6 to 8 ft (1.8 to 2.4 m) depth may be assumed for planning purposes.

Stormwater Run-off Control

Slow rate land treatment systems must usually provide some method of control of stormwater run-off to prevent erosion. Sediment control may be achieved by use of sediment control basins, or control measures such as terracing of steep slopes, grass border strips, and stream buffer zones. Since the slow rate systems proposed for the study area utilize a grass forage crop, rather than crops requiring plowing and exposure of the soil to the elements, it is felt that sediment control will not be a great problem. Since wastewater application is not allowed during times when storm run-off might occur, recirculation of the storm run-off for further treatment is not considered necessary (4).

Due to the relatively flat slopes and the high permeabilities of soils used for rapid infiltration land treatment, surface run-off is not considered to be a problem.

System Monitoring

Slow Rate — Monitoring facilities will be required to determine groundwater quality both before wastewater application begins (base line data) and during wastewater application by the slow rate treatment process. Sampling wells around the perimeter of the application site and within the application site itself, will be required to determine the performance of the land treatment process. The wastewater being applied to the soil must be monitored, as well as any percolate collected for discharge to area streams. The surface water discharge must be monitored to insure compliance with water quality standards and NPDES permit requirements. Typical monitoring requirements can be obtained from Reference (4).

Rapid Infiltration — Since the effluent from the rapid infiltration land treatment system will be collected and discharged as a point source to surface waters, NPDES permit requirements must be satisfied. It may also be necessary to disinfect this effluent prior to discharge to the surface water. Costs for rapid infiltration systems presented in a later section include disinfection. Monitoring requirements for rapid infiltration systems include analysis of the wastewater prior to application, as well as effluent analysis. Sampling wells are generally also required to insure that the percolate is being collected by the underdrain or recovery well system, and that groundwater quality is not being degraded.

Land Treatment System Costs

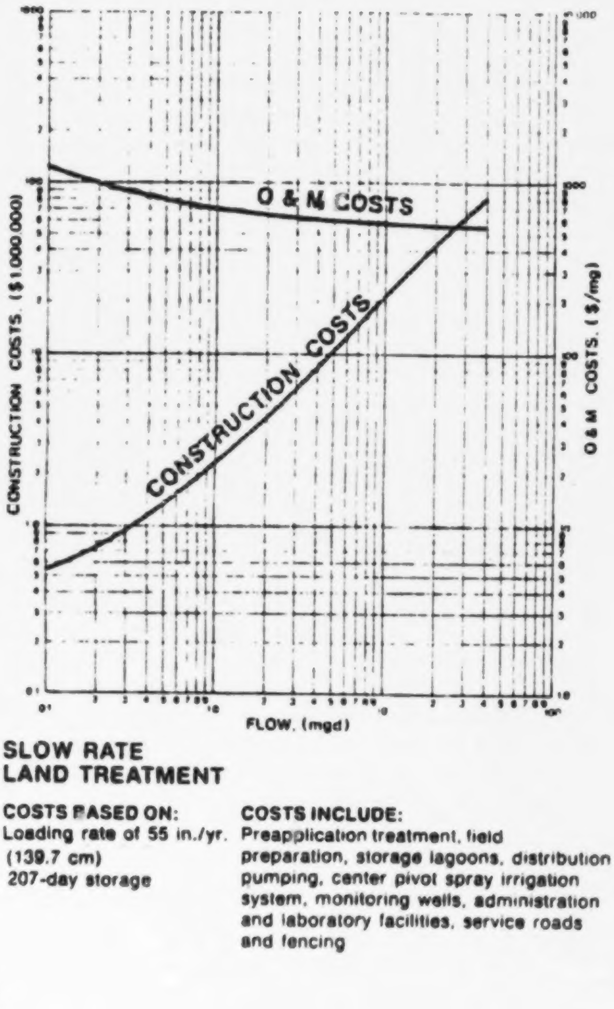
General

Costs for construction and operation and maintenance of land treatment systems proposed for use in the study area have been developed. These costs are presented in the cost curves on Figures 3 and 4. In addition, cost information for transmission of wastewater to the land treatment sites has been developed.

Cost Curve Development

Construction and operation and maintenance (O&M) costs were developed for each component of the land treatment systems. The costs for the individual components were then aggregated by system to arrive at the total land treatment system cost curves presented on Figure 3. Most of the unit costs for individual components were based on Reference (27). The time base for all curves is June 30, 1977. Indices and unit costs utilized in cost curve development are as follows:

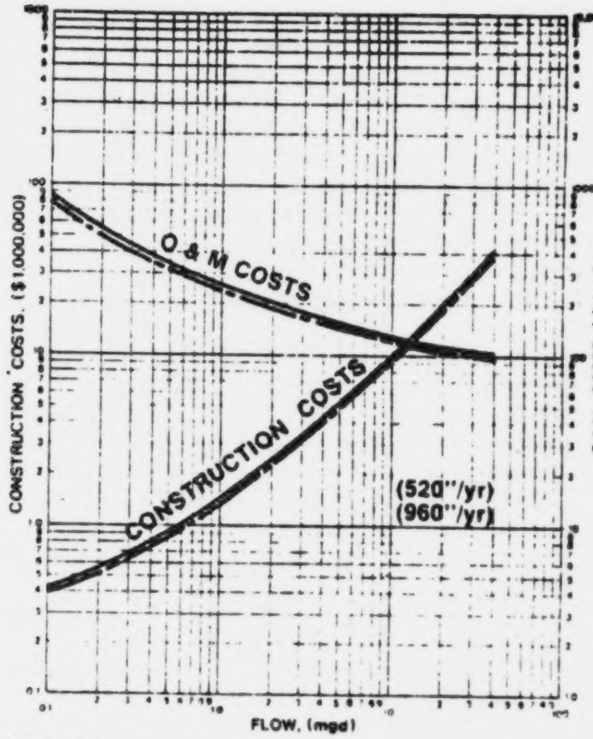
EPA Sewage Treatment Plant Construction Cost Index (LCAT), Minneapolis, Second Quarter, 1977	127.0
EPA Sewer Construction Cost Index (CUSS), Minneapolis, Second Quarter, 1977	133.0
Wholesale Price Index, Industrial Commodities, June 30, 1977	194.6
ENR Construction Cost Index, Minneapolis, June 30, 1977	2575.8
Treatment System Operation and Maintenance Labor	$9.50/hr
Electricity	2.5¢ kWh



SLOW RATE
LAND TREATMENT

COSTS BASED ON:
Loading rate of 55 in./yr.
(139.7 cm)
207-day storage

COSTS INCLUDE:
Preapplication treatment, field preparation, storage lagoons, distribution pumping, center pivot spray irrigation system, monitoring wells, administration and laboratory facilities, service roads and fencing

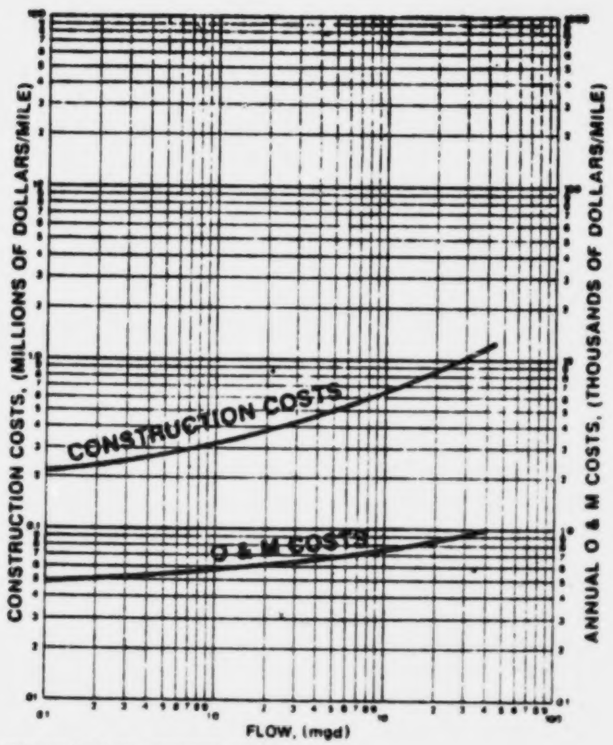


RAPID INFILTRATION LAND TREATMENT

COSTS BASED ON:
120-day storage, loading rates as shown

COSTS INCLUDE:
Preapplication treatment, field preparation, storage lagoons, distribution pumping, infiltration basins, recovery wells, effluent disinfection, monitoring wells, administration and laboratory facilities, service roads and fencing

Figure 3
Cost Curves



GRAVITY INTERCEPTOR

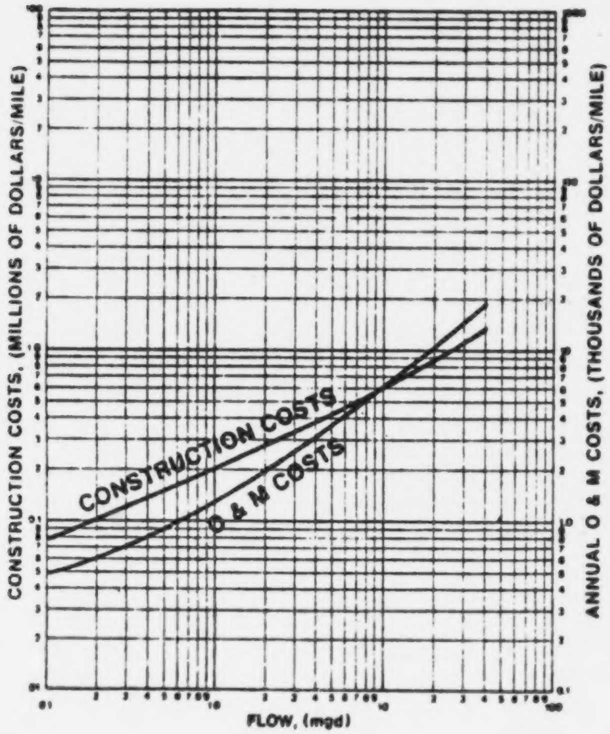
COSTS BASED ON:
Trench depths of 6'-28', average depth of 17'

Minimum velocity = 2.5 feet per second at design flow

Manholes spaced at 350' (8"-60") and 500' (> 60")

Peaking factors vary from 3.0 @ 1 mgd flow to 2.0 @ flows ≥ 10 mgd

COSTS INCLUDE:
Material, layback trenching, labor



FORCE MAIN - PUMPING STATION WASTEWATER TRANSMISSION SYSTEM

COSTS BASED ON:
6-foot depth of cover over pipe crown

Average velocity approximately 5 feet per second

Total pumping head = 200 feet

Peaking factors vary from 3.0 @ 1 mgd flow to 2.0 @ flows ≥ 10 mgd

COSTS INCLUDE:
Force main - material, layback trenching, labor

Pumping station - valves, controls, wet well, dry well, and enclosing structure

Figure 4
Cost Curves

Slow Rate Treatment Cost Curves

Items for which costs for construction and operation and maintenance were figured individually to form the composite cost curve for the slow rate land treatment system are presented on the following page.

1. Preapplication treatment consisting of preliminary treatment (screening, grit removal, and metering) and an aerated lagoon of 6 days detention.
2. Storage lagoon of adequate volume to hold 207 days flow.
3. Site clearing of the land treatment site. Costs based on an area of mostly brush with a few trees, and clearing using bulldozer-type equipment.
4. Heavy-duty electrically driven center pivot spray rig for wastewater application.
5. Distribution pumping consisting of a pumping system providing 250 ft (76.2 m) of head. Capital costs are related to the peak flow supplied to the distribution system while O&M costs are based on average flow.
6. Administrative and laboratory facilities.
7. Monitoring wells.
8. Service roads and fencing.
9. Planting, cultivation, and harvesting costs based on reed canarygrass crop.

The cost curves do not include:

1. The cost of land.
2. Costs of transmitting the wastewater from the collection system to the land treatment site.
3. Costs for relocation of residents who are living in the proposed land application area.
4. Benefits (negative costs) due to the harvest of reed canarygrass crop.
5. Engineering, legal, administrative fees, interest during construction, or contingencies.

Rapid Infiltration Treatment Cost Curves

Items included in the cost curves for rapid infiltration systems are as follows:

1. Preapplication treatment (same as for slow rate system).
2. Storage lagoons of sufficient volume to hold 120 days flow.
3. Site clearing of area with brush and a few trees using bulldozer-type equipment.
4. Multiple-unit infiltration basins with 4 ft (1.2m) dikes.
5. Distribution pumping system with pumping head of 50 ft (15.2 m) to pump from the treatment or storage lagoons to the infiltration basins.
6. Recovery wells using vertical turbine pumps with associated electrical work and shelter.
7. Chlorination of rapid infiltration treatment effluent prior to discharge to surface waters.
8. Administrative and laboratory facilities.
9. Monitoring wells and effluent sampling systems.
10. Service roads and fencing.

The cost curves do not include:

1. The cost of land.
2. Costs of transmitting the wastewater from the collection system to the land treatment site, or of transmitting the effluent from the recovery wells to the point of discharge to surface waters.
3. Costs for relocation of residents living in the proposed land application area.
4. Engineering, legal, administrative fees, interest during construction, or contingencies.

Transmission System Costs

Costs for typical transmission systems, both gravity and force main, have been developed for various quantities of flow. Construction and O&M cost curves for transmission systems are shown on Figure 4.

Gravity Pipelines — Gravity transmission system cost curves are based on the following design critera:

1. Trench depths range from 6 to 28 ft (1.8 m to 8.5 m), average depth of 17 feet (5.2 m).
2. Minimum velocity at the design flow is 2.5 ft/sec (0.76 m/s).
3. Manning's roughness coefficient is assumed at $n = 0.013$.
4. Manholes are spaced at 350 ft (107 m) for 8 to 60 in. (20.3 to 152.4 cm) while lines ≥ 60 in. (≥ 152.4 cm) in diameter have manholes spaced 500 ft (152 m). The average depth of manholes is 17 ft (5.2 m).
5. Pipe slope varies from 0.05 ft/100 ft to 0.25 ft/100 ft.

Force Mains — Typical design criteria used for costing of force mains are as follows:

1. The depth of cover over the crown of the pipe is 6 ft (1.8 m).
2. Minimum velocity in the force main is 2 ft/sec (0.61 m/s), with a maximum velocity of 8 ft/sec (2.44 m/s).
3. Pumping total dynamic head is 200 ft (61 m).
4. The Hazen-Williams coefficient is $C = 120$.

Costs for the force main system include force main piping and a pumping station with wet well, dry well, valving, site work, and standby equipment.

Construction Costs — Neither gravity nor force main construction cost curves include contingencies, engineering and legal fees, administrative costs, or interest during construction.

Both gravity and force main systems are sized to handle the peak flow, utilizing peaking factors common to the area (5). Costs associated with both force main and gravity sewer transmission line construction include allowances for clearing and grubbing, roadway repair and surface restoration, as well as normal dewatering costs. Special construction techniques or requirements due to specific problems encountered in the field (crossing major roadways, crossing major rivers or streams, very deep pipelines requiring extensive excavation or tunneling, rock excavation, extensive dewatering, etc.) would require special cost calculations.

Operation and Maintenance Costs — O&M costs for both gravity and force main systems include the cost of periodic cleaning by a contractor, while force main systems also include the costs associated with pumping station operation and maintenance (power, labor for operation and repair, and replacement parts).

Cost Curve Usage

Capital Cost Determination — The capital, or construction cost, of a proposed land treatment system may be calculated by summing the following four cost items. Costs thus calculated should be used for preliminary planning purposes only. Actual total capital costs will vary from site to site and require a more detailed site specific cost analysis. For preliminary planning purposes and comparison of land treatment alternatives, however, the cost methodology proposed herein should be sufficiently accurate.

1. Based on the flow projected at the end of the planning period, the construction costs for the land treatment system components can be calculated by entering the appropriate land treatment system composite construction cost curve, Figure 3, and obtaining the corresponding construction cost. This cost should be multiplied by a service and interest factor of 1.27 to include costs of engineering, legal, administration, contingencies, and interest during construction.
2. Again, based on the flow to be treated at the end of the planning period (or whatever ultimate flow is considered appropriate for pursuit of land acquisition), the total land area required for the land treatment system can be determined. Based on knowledge of land cost in the area of the proposed treatment site, the total land acquisition costs may be determined by multiplying the number of acres (ha) needed by the $/acre ($/ha). Total land requirements for the various systems in relationship to flows were presented under "LAND TREATMENT DESIGN CRITERIA."
3. The cost of transmission systems to convey the flow from the existing municipal collection system to the land treatment site may be calculated using the transmission system cost curves presented on Figure 4. Use of USGS maps will allow general determinations to be made of whether force main or gravity transmission lines are required, and the approximate length needed. Based on this data, the approximate costs for transmission of wastewater to the land treatment site may be obtained. Land treatment systems discharging to surface waters (rapid infiltration systems and slow rate systems where groundwater mounding is a problem) will also require transmission systems to convey the flow to the ultimate point of discharge. Again, USGS maps may be utilized to make preliminary decisions regarding length and type of transmission system required. Construction costs of transmission systems from the cost curves must also be multiplied by the 1.27 service and interest factor, as discussed under Item 1.
4. Costs must be evaluated and included for relocation of families living in the proposed land treatment system site. Local agencies such as the Minnesota Highway Commission and the St. Paul District, U.S. Army Corps of Engineers, may be able to provide additional information regarding these costs.

Operation and Maintenance Costs — The yearly cost for operation and maintenance of land treatment systems may be calculated by addition of the following factors:

1. Operation and maintenance costs for the land treatment components may be calculated by entering the appropriate composite land treatment system cost curve and obtaining the O&M cost in $/mil gal ($\$/m^3$) for the appropriate flow being treated in the year under consideration. This cost /$/mil gal ($\$/m^3$)/ is then multiplied by 365 times the flow in mgd (m^3/d) to obtain a total annual cost.
2. Operation and maintenance of transmission systems are associated with O&M costs for pipelines and pumping stations. Again, from the transmission system cost curves on Figure 4, an annual cost for transmission system O&M can be calculated by multiplying the length of the transmission system in miles (km) by the annual $/mile ($/km) from the appropriate cost curve.
3. Slow rate systems will harvest reed canarygrass three times during the application season. This grass can probably be sold, resulting in a total dollar value that should be subtracted from the annual cost for operation and maintenance obtained in Items 1 and 2.

Comparison of Alternatives

Comparison of land treatment alternatives to determine the most cost-effective alternative for wastewater management should be done in accordance with EPA cost-effectiveness analysis procedures contained in 40 CFR 35. The interest rate to use in this analysis is determined by the Water Resources Council, and is currently valued at 6 5/8 percent. Either the present worth or equivalent annual cost method of analysis may be used to place alternatives on a common basis for evaluation.

Assuming the present worth method of analysis is utilized, it is necessary to determine constructin costs for proposed land treatment systems and transmission system components, as well as the value of the remaining economic life of the systems at the end of the planning period (salvage). Land is assumed, per the cost-effectiveness guidelines, to have a value at the end of the planning period equal to its current market value at the time of purchase. Gravity pipes and force mains are assumed to have economic lives of 50 years, while pumping stations can be assumed to have a life of 25 years. Individual components of the land treatment systems have varying service lives, but an overall composite service life for the complete land treatment system of 30 years may be utilized for preliminary planning purposes

(28). Relocation costs are a one-time cost at the start of the study period. By knowing the times when cash expenditures for the various items are made and the remaining economic life and salvage values of land, treatment systems, and transmission systems, it is possible to arrive at the present worth cost of construction for the proposed system.

Yearly operation and maintenance costs (less crop harvest value) may be calculated as described in the previous subsections. These costs may be discounted back to the base year by application of the appropriate factors.

The total present worth value of an alternative is equal to the total present worth construction cost plus the total present worth of operation and maintenance.

Land Treatment Site Screening Methodology

General

Identification of potential sites for land treatment in the study area required analysis of land use (existing and future), soils, slopes and surface and groundwater conditions. Criteria were established for each of these parameters and the study area was evaluated on the basis of each parameter in turn. If an area was considered unacceptable on the basis of one of the parameters, it was ruled out and not evaluated for suitability of remaining parameters.

The result of the screening is presented on the Figure 5 map. Those areas shown on the map as potential land treatment sites are based on the site screening methodology used in this study. Field investigations will be required to positively confirm that a specific site has all the necessary characteristics required for the desired type of system and to confirm specific design requirements.

Land Use

Land use screening was conducted to exclude those areas deemed unsuitable for land treatment processes. Computer generated maps based on the Minnesota Land Management Information System (MLMIS) were used as an aid to screening. The MLMIS divides land use into nine distinct classifications: forested, cultivated, water, marsh, urban residential, extractive, pasture/open space, urban/non- or mixed residential, and transportation. In addition to the MLMIS maps, USGS maps and maps prepared for other studies were used (16, 17).

The initial screening process was performed using map overlays with the MLMIS map to exclude urban residential, urban/non- or mixed residential and extractive land uses. Subsequent screening using USGS maps identified additional areas encompassing these land use classifications that were excluded. Using maps provided by the Metropolitan Council, urban expansion to the year 2000 was delineated as shown on the final map presented on Figure. 5.

Many areas shown on the Figure 5 map are designated as existing built-up areas but are outside the year 2000 Metropolitan Urban Service Area (MUSA) boundary. These areas generally consist of residential or mixed residential land uses bordering the MUSA, but also include free standing growth areas and small developments.

Identifiable parks, wildlife preserves, and recreation areas were excluded as a part of this use classification.

Within the areas classified as suitable for land treatment are areas of scattered forest. It may be necessary to clear some of these forested areas or design around them when specific land treatment sites are planned. Also, within the areas classified as suitable, there are transportation facilities such as roads, highways, and railroads. These facilities will require consideration when siting specific land treatment systems.

Soils

The physical characteristics of the soil directly influence the suitability of an area for land application. Soil properties requiring evaluation include texture and structure, infiltration rate and permeability, moisture-holding capacity, and soil depth. The position of the soil in the landscape can also be a significant factor in determining its suitability for land treatment.

The soil infiltration rate and permeability depend on the texture and structure of the soil. Medium-textured (loamy) soils are best for slow rate systems while coarse-textured (sandy) soils are best for infiltration-percolation systems. Well structured soils are desirable because the large pores readily conduct air and water within the soil. The infiltration rate must be sufficiently high to minimize the occurrence of effluent ponding and the associated run-off hazard. The permeability of the most restrictive soil layer in the upper 5 ft (1.5 m) of the soil profile should equal or exceed the infiltration rate to facilitate downward movement of applied effluent in order to prevent the root zone soil from becoming saturated for long periods.

Soil moisture-holding capacity is an indication of the soil's textural class. Fine-textured soils have high moisture-holding capacity and are not readily drained. Such soils are generally unsuited to land application.

Soil depth is an important factor in land application. Plant growth, available soil moisture, and treatment time in the soil are all functions of soil depth. Where shallow soil depth is underlain with fractured or crevassed rock, the applied effluent may be short-circuited to the underlying groundwater before it has been adequately treated in the soil.

Soils information was obtained from Soil Conservation Service county soil surveys and other studies (11, 12, 13, 14, 15, 16, 17). Table 8 presents those soil characteristics considered most desirable for the potential

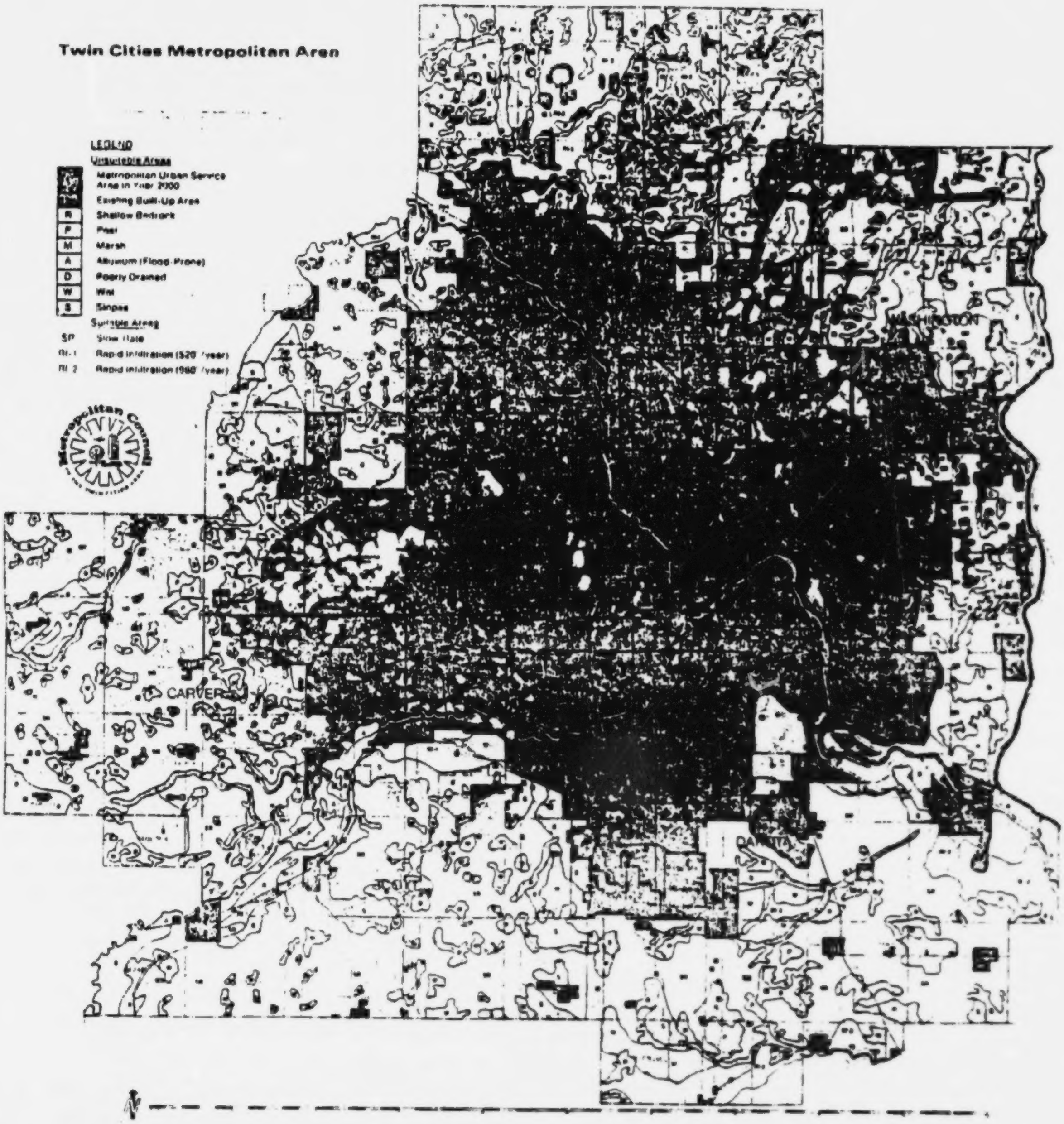

Twin Cities Metropolitan Area
LEGEND
Unsuitable Areas
Metropolitan Urban Service Area in Year 2000
Existing Built-Up Area
R
Shallow Bedrock
P
Peat
M
Marsh
A
Alluvium (Flood-Prone)
D
Poorly Drained
W
Wet
S
Slopes
Suitable Areas
SR
Slow Rate
RI-1
Rapid Infiltration (520'/year)
RI-2
Rapid Infiltration (980'/year)
Metropolitan Council
CARVER
WASHINGTON
DAKOTA

Figure 5
Potential Land Treatment Sites in the Twin Cities Metropolitan Area, Minnesota

land treatment processes evaluated herein.

Table 9 presents soil characteristics and classifications deemed unsuitable for use in a land treatment system.

Although alluvial soils subject to flooding were screened out in this analysis, it might be possible to locate wastewater application areas /"wet acres (ha)"/ in the flood plain. Rapid infiltration systems are probably best suited to the soils found in these areas and do not require mechanical center pivot spray rigs which might be damaged by flooding. The final decision as to suitability of these areas is dependent on the frequency and severity of flooding and can only be determined by a detailed analysis of specific sites. It would be required that storage facilities, office and lab areas, and preapplication treatment systems be located out of the area subject to flooding. Storage lagoons must be adequate to store the volume of wastewater received at the site during times when the application area was flooded and whatever time was required after flood subsidence before wastewater application could be resumed.

Table 8

Desirable Soil Characteristics for Land Treatment

Soil Characteristic	**Land Treatment Process** **Slow Rate**	 **Rapid Infiltration**
Minimum Depty	5 ft^3	5 ft^3
Texture	medium (loam to silt loam)	moderately coarse (sandy loam) to coarse (loamy sand)
Permeability	moderate (0.60-2.0 in./hr)^4	moderately rapid (2.0-6.0 in./hr)^4 to rapid (6.0-20 in/hr)
Depth to Ground water	5 ft^{1,3}	10 ft^{1,3}
Slope	$\leq$15%	$\leq$5%^{2}

Notes: [1] Shallower depths are acceptable where underdrainage is provided.
[2] Steeper slopes will require progressively greater earthwork operations during construction. If infiltration basins are terraced into slopes, lateral movement of water from upper basins may affect percolation rates in lower basins.
[3] 1 ft = 0.3048 m.
[4] 1 in./hr = 2.54 cm/h.

Source: Stanley Consultants, Inc.

Table 9

Soil Characteristics and Classification Unsuitable for Land Treatment

Rock (R)	Shallow depth to bedrock, possible inadequate soil depth. May result in groundwater mounding or may short circuit aplied effluent to groundwater aquifers without adequate treatment if rock is extensively fractured.
Peat (P)	Organic soils formed in low, wet areas. Unsuitable due to wetness in its natural state.
Marsh (M)	Unsuitable due to wetness in its natural state.
Alluvium (A)	Soils along stream channels susceptible to flooding.
Poorly Drained (D)	Soils with permeability rates unsuitable for use in land treatment systems.

Wet Soil Conditions

There are a large number of lakes and wet depressions associated with the broad physiographic moraine areas in the metropolitan counties. Many of the depressions in these moraine areas are subject to periodic inundation due to the seasonal high water table. Often these depressions are closed; i.e., the drainage system is not integrated and the depressions cannot be easily drained.

Land treatment of wastewater effluent is predicated on the fact that the soil and cover crop act as a treatment medium. A soil depth of 3 to 5 ft (0.9 to 1.5 m) is necessary to provide adequate treatment before the percolating effluent reaches the groundwater table. The screening process attempted to eliminate those areas where the groundwater table was near the surface. Screening was accomplished using USGS maps, a water table elevation contour map (18), and maps prepared for other studies (16, 17, 26).

It should be noted that scattered areas deemed unsuitable as a result of areawide screening may be found suitable when evaluated on a site specific basis. Such areas may be made suitable through installation of underdrains or recovery wells. This is particularly true in the Anoka Sand Plain geomorphic region.

Slopes

Within the study area there are a wide variety of topographic conditions ranging from nearly level to gently rolling to steeply sloped. Steep slopes are more susceptible to erosion hazards and are likely to produce run-off conditions. For this study, areas with slopes in excess of 15 percent are considered unsuitable for land application. Screening on an areawide basis was conducted to identify and eliminate significant areas where unsuitable slope combinations are present. Screening was accomplished using USGS quadrangle maps and Soil Conservation Service Soil Survey books (11, 12, 13, 14, 15).

Within the areas designated as being generally suitable for land application there may occur small tracts with slopes greater than 15 percent which are difficult to locate in areawide screening. These small areas will have to be identified and dealt with when a land treatment facility is being considered for a specific site.

Land Treatment Site Selection and Costing Example

General

This section presents a preliminary analysis of a potential site and present worth cost of land treatment for approximately two-thirds of the flow from the Lake Elmo (Tri-Lakes and Old Town) area. The projected flow to be treated, the location of the sources of wastewater input to interceptor systems for transmittal to the land treatment site, and general location of a site were furnished by the Metropolitan Council. This basic input was used to locate and layout a potential site to handle the projected flow in the year 2000 and do a preliminary design of the wastewater transmission system required to carry the flow from the service area to the site. Construction costs and O&M costs for the land treatment process have been calculated and are presented on a present worth basis.

Flow projections for the area to be served by land treatment are as follows:

1980: 0.24 mgd (908 m^3/d)
1990: 0.30 mgd (1,136 m^3/d)
2000: 0.73 mgd (2,763 m^3/d)
2020: 1.24 mgd (4,693 m^3/d)

Treatment Process Design

The slow rate process has been selected for land treatment of the Lake Elmo flow. Soils in the area of the proposed site have general permeabilities in the range of 0.6 to 2.0 in./hr (1.5 to 5.1 cm/h), with slopes generally 13 percent or less. Soil series contained in the application site are shown in Table 10.

General design criteria proposed in earlier sections of the report for slow rate systems are utilized for this system and are presented on the following page.

1. Apply 55 in. (140 cm) of wastewater per year over a 26-week period, starting in late April and continuing through the middle of October. Weekly application rates would be similar to those shown in Table 5.
2. Reed canarygrass would be grown on the application site, with harvest three times per season.
3. Preapplication treatment would consist of preliminary treatment and 6 days detection in an aerated lagoon.
4. A storage lagoon of sufficient volume to hold 207 days flow would be required.
5. Wastewater application would be accomplished by center pivot spray irrigation rigs.
6. Percolate will discharge to the groundwater. No additional cost for subsurface drainage is included.
7. Approximately 8 monitoring wells have been assumed. These wells would be located both on-site and adjacent to the land treatment site.
8. Buffer zones of 200 ft (61 m) are provided adjacent to roads or other areas where people might be found. Some areas abutting on the airport property do not require a full 200 ft (61 m) buffer width. The entire land treatment site would be fenced.

Land Area Requirements

Application of 55 in. (140 cm) of wastewater per year requires 244 "wet acres (ha)" per mgd (0.026 $ha/m^3/d$) of wastewater. Assuming that the site will be designed to handle the year 2000 flow of 0.73 mgd (2,763 m^3/d), the "wet acres (ha)" required is approximately 180 acres (73 ha).

The aerated lagoon, based on a 15 ft (4.6 m) depth and 3 to 1 side slopes, will occupy approximately 1.6 acres (0.65 ha). The storage lagoon with a 12-ft (3.7 m) depth, 3 to 1 side slopes, and two cells with require an additional 45 acres (18.2 ha).

The actual layout of the system is shown on Figure 6. The total area required for the land treatment site is greater than the 350 acres/mgd (0.037 $ha/m^3/d$) proposed for planning purposes due to the roadways and other constraints inherent in the general location originally set forth for siting analysis. Total area required for the site is 400 acres (162 ha).

Figure 6 shows tentative locations of the treatment and storage lagoons, as well as a possible configuration for deployment of center pivot spray rigs.

Wastewater Transmission System Design

Wastewater transmission systems were designed to handle the 2020 flow. This flow was utilized to be consistent with transmission system costs developed for mechanical wastewater treatment facility alternatives developed in a previous study (5). Land treatment of the Lake Elmo flow was one of the alternatives originally delineated for analysis in that study. In addition, some land is available in the vicinity of the proposed site which might be used for expansion of the process after the year 2000. The wastewater transmission system could be extended at that time to serve an adjacent site.

The flow from the Tri-Lakes area (one-half of projected flow in "General" subsection) would be pumped through a 12,500 ft (3,810 m) long, 10 in. (25.4 cm) diameter force main to a point where it would be combined with the flow from the Old Town area (remaining one-half of projected flow). The Old Town area flow would also require a pumping station to convey its flow through the 2,500 ft (762 m) long, 14 in. (35.6 cm) diameter force main carrying the total combined flow from the point of addition to the proposed land treatment site. Force main systems were selected due to the great pipe depths needed for gravity flow. Gravity flow would have required special "open cut" construction through many areas where normal trenching machinery would not provide adequate pipe depth. Proposed pumping station and pipe locations are shown on Figure 6.

Land Treatment System Costs

On-Site Land Treatment System — The construction cost of the components of the on-site land treatment system may be obtained from Figure 3. For a flow of 0.73 mgd (2,763 m^3/d), the approximate construction cost is $1.87 million. Multiplying by the service and interest factor of 1.27, the capital cost of the land treatment system at the time of construction in 1979 would be approximately $2.37 million. The value of the remaining economic life (salvage) at the end of the study period (year 2000), based on straight-line depreciation over a 30-year economic life, is $0.79 million in the year 2000. Discounting both the construction cost in 1979 and the value of the remaining economic life in the year 2000 back to the base year of 1977 using a 6 5/8 percent interest factor results in a total present worth of construction (construction present worth less salvage present

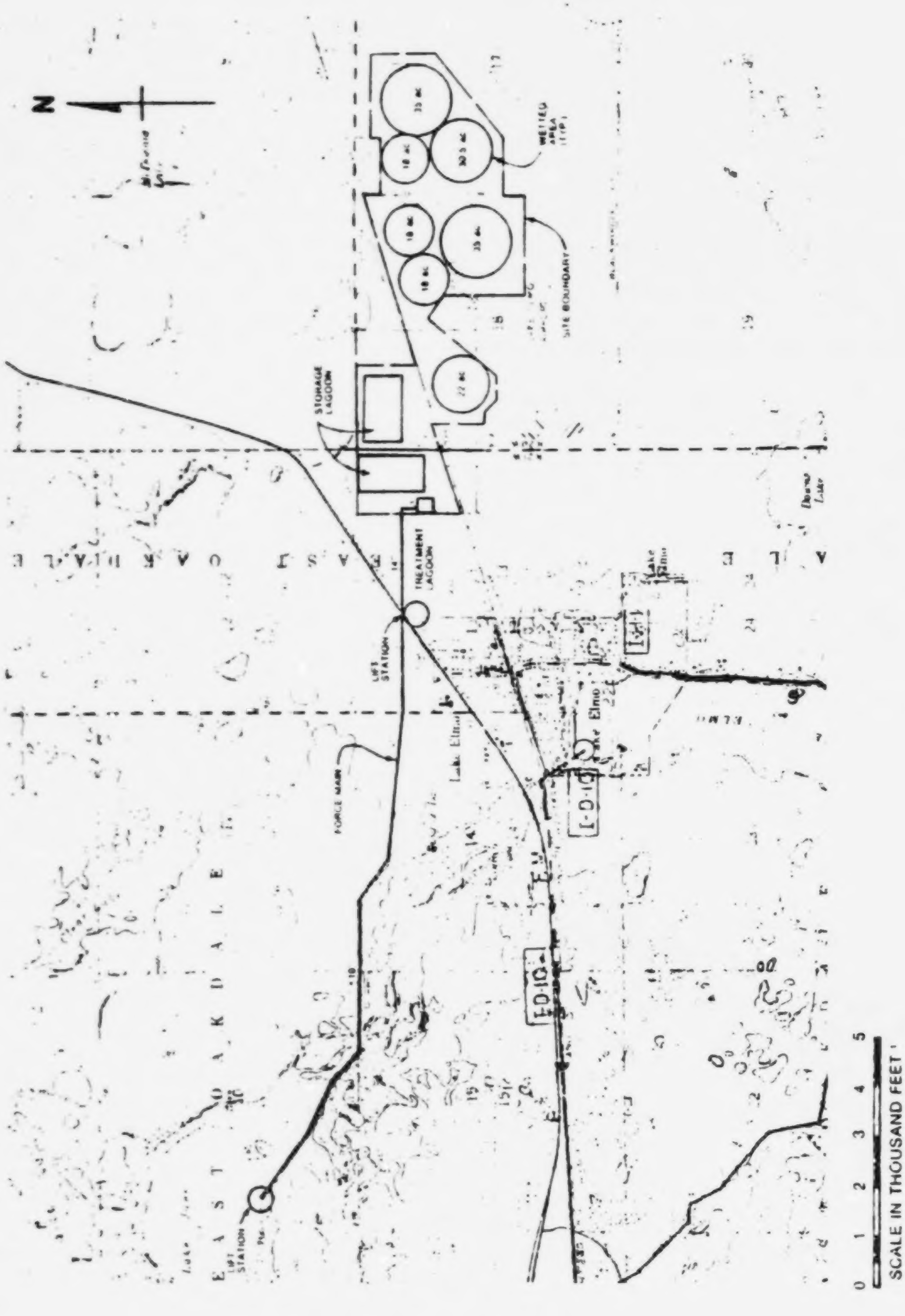

N
STORAGE LAGOON
TREATMENT LAGOON
LIFT STATION
FORCE MAIN
WETTED AREA (TYP.)
SITE BOUNDARY
EAST OAKDALE
Lake Elmo
0 1 2 3 4 5
SCALE IN THOUSAND FEET

Figure 6
Proposed Lake Elmo Land Treatment Facility

Table 10

Soil Types and Physical Properties for Potential Lake Elmo Land Treatment Site

			Most Comon Texture and Thickness [ft(m)]				Moisture Relationships		
County	Geomorphic[1] Region	Significant Soil Series	Root Zone	Substratum	In of High Water Table (ft)[3]	Slope Range (Percent)	Available Water to 5 ft[4]	Drainage Class	Permeability[2] (in./hr)[5]
Washington	Mississippi Valley Outwash	Waukegon	Silt Loam (2-3)	Sand & Gravel	>5	0-8	7-9	Well	0.6-2.0
		Dakota	Sandy Loam (2-3)	Sand	>6	8-13	6-9	Well	0.6-2.0
		Dakota	Loam (3 +)	Sand	>5	3-8	7-9	Well	20-6.0
		Judson	Silt Loam (3)	Silt Loam	4-6	0-3	8-11	Well to Moderately Well	0.6-2.0

Notes: [1] From Reference (16).
[2] Permeability of most restrictive layer in upper 5 feet.
[3] 1 ft = 0.3048 m
[4] 1 in. = 2.54 cm
[5] 1 in./hr = 2.54 cm/h

Sources: References (16, 17, 18) and Stanley Consultants, Inc.

worth) of $1.90 million.

Operation and maintenance costs will vary each year of the study period as the flow increases. O&M present worth costs are most easily calculated by assuming a straight-line increase in flow between dates for which flow is calculated and use of annual and gradient series to arrive at present worth costs. The O&M costs (annual costs) for the land treatment system components for years 1980, 1990, and 2000 would be approximately $0.087, $0.101, and $0.2 million, respectively. The total present worth cost of O&M of the treatment system for the study period of 1980 to 2000 (base year of present worth analysis is 1977) is $0.97 million.

Wastewater Transmission System Cost — The total cost for constructing the wastewater transmission line and pumping stations (including service and interest factor) in 1979 has been estimated at a cost of $1.17 million. This cost is based on a layout of the pipe routing on USGS maps and cost estimation based on pipe length utilizing cost data developed in a previous study (5). Pumping station costs include cost of a wet well/dry well structure, pumping equipment, internal piping and valves, controls, and electrical work.

The present worth of the construction cost of the force main and pumping stations is $1.03 million. Assuming economic lives of 50 years for the pipe and 25 years for the lift stations results in a total present worth of construction (initial cost less salvage) of $0.93 million for the transmission system.

Operation and maintenance costs for the transmission system will vary yearly with flow. The present worth of O&M for this sytem, based on transmission pumping O&M costs developed in a previous study (5), is $0.054 million.

Total Present Worth Cost — Summing up the total present worth costs of construction and O&M for the on-site land treatment system and wastewater transmission system results in a total present worth value of $3.85 million. This cost does not include the present worth of land acquisition (initial purchase less salvage), the one-time cost of family relocation, or any benefits resulting from marketing of the reed canarygrass crop.

The total present worth value of $3.85 million for a land application system should be compared to alternate mechanical wastewater treatment systems that achieve equivalent effluent standards to determine if land application is cost-effective for the Lake Elmo area.

Summary

Criteria for preliminary design and costing of slow rate and rapid infiltration land application systems in the Minneapolis-St. Paul, Minnesota area have been developed. Using these criteria, typical construction and operation and maintenance costs for land application systems have also been determined. Climatological, land use, soils, and topographic data has been used to identify potential land application sites in the study area. An example of how this information can be used to preliminarily design and cost a land application system was presented.

The Metropolitan Council of the Twin Cities has used the information and methodology presented in this paper to investigate a variety of potential land application systems in the Minneapolis-St. Paul area. At this time, several land application systems in the study area appear to be potentially cost-effective. These potential systems are for smaller sized communities on the outskirts of the metropolitan area and appear to be cost-effective when tertiary wastewater treatment is required. They will be investigated in greater detail during the 201 Facility Planning Studies that are currently under way.

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The Mathematical Model of the Adsorption of Organic Impurities from Water Solutions with Microporous Adsorbents

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The analysis of the Soviet and foreign literature shows that the adsorption technology is the most perspective and economically expedient for the deep purification of natural and waste waters from dissolved organic pollutants. In this connection the practical problems of design and optimization of the adsorption treatment of water as well as the construction of the highly effective sorption equipment demands the appropriate development of the theory of solutes adsorption. However, both the choice of sorbents for these purposes and the design of sorption units were mainly based up to now upon the purely empiric approach using empiric constants, the changes of which while passing from one system to another couldn't be predicted. This necessitated carrying out numerous experiments in every concrete case first in a laboratory and then on a pilot plant.

The attempts to create the formalized mathematical models of the adsorption treatment of wastewaters also do not seem to be a solution of the problem since such models again use empiric constants.

The aim of our investigations was the development of the theoretical basis for a mathematical model of adsorption of organics (partly soluble in water) from water solutions with microporous adsorbents under static and dynamic conditions (for a stationary adsorbent bed). We mean such a mathematical model, which is based on the adequate reflection of physico-chemical processes taking place when organic substances are adsorbed and which permits to carry out the design of adsorption units for water treatment, to choose the most effective sorbent for given conditions and to evaluate the technical and economic efficiency of the process, the experimental data being minimal.

The description of the process of adsorption from solutions under equilibrium and non-equilibrium conditions is the basis of the suggested model. Let us begin with the description of the adsorption equilibrium. According to the theory of the volumetric filling of micropores, developed by M. M. Dubinin and L. V. Radushkevich /1/ the equation of the characteristic adsorption curve may be presented in the form:

$$W = W_0 \exp\left\{-K\epsilon^2\right\} \tag{1}$$

This Equation expresses the distribution of the filled volumes of the adsorption space W along the differential molar works of adsorption ϵ. W_0 is the maximum volume of the adsorption space, characterizing the volume of the adsorbent's micropores; K — is the constant, characterizing the adsorption system. This Equation was obtained for the case of vapours adsorption on microporous activated carbons. In our work /2/ we suggested that this Equation should be applied to the case of adsorption from solutions on the ground that the function of the adsorbent pores distribution in principle must not depend on whether adsorption occurs from gaseous or liquid phase.

On the basis of the thermodynamics of the maximum diluted solutions (such are solutions of many organic substances partly soluble in water) it is shown that the differential molar work of adsorption may be expressed in this case as:

$$\epsilon = RT\ln C_s/C_p \tag{2}$$

where C_s is the concentration of a solute, corresponding to the saturated solution;

C_p — the equilibrium concentration of a solute in the solution;

R — gas constant;

T — absolute temperature.

After substitution of Equation 2 into Equation 1 and some transformations we obtain the Equation of the adsorption isotherm:

$$\log \Gamma_p = \lg \frac{W_0}{V_m} - 2.303\,(\lg \frac{C_s}{C_p})^2 \cdot \frac{BT^2}{\beta^2} \tag{3}$$

Where Γ_p — specific equilibrium adsorption;
V_m — molar volume of the adsorbate;
B — structural-energetic constant, characterizing the adsorbent;
β — affinity factor expressed by the relative molar work of adsorption of the given solute at a selected standard substance.

The relation between constants K and B is expressed as:

$$B = KR^2\beta^2 \tag{4}$$

It is obvious that if the experimental adsorption isotherm satisfies Equation 3, then in logarithmic coordinates lg Γp; $(\lg^{C_s}/C_p)^2$ it must be the straight line with the angle of inclination tangent tg $\alpha = 2{,}303$ BT^2/β^2, cutting a segment $\lg W_o/V_m$ on the ordinate axis. The ratio W_o/V_m is the maximum specific adsorption. For a standard substance the affinity factor is assumed to be equal to 1. Benzol in a vaporous state may be chosen as the latter. According to the given model, each adsorbent is characterized by two parameters: W_o and B. If an adsorbent possesses the bidispersal microporous structure, each of its substructures will be characterized by its own values (W_{o_1}, B_1) and (W_{o_1}, B_2).

In accordance with Equation 3 the processing of data on adsorption equilibrium consists of the linearization of adsorption isotherm in logarithmic coordinates in order to obtain the corresponding equilibrium characteristics for each adsorbent under investigation.

Up to now the constant B has been determined from the adsorption measurements only. It has been noted that its value is related with the size of activated carbon micropores: the smaller is the micropores size, the lower is the value of B. However, the evident form of this relationship has not been obtained. In this connection it is of indubitable interest to obtain the constant B by the independent way and to find the evident form of the function mentioned. In work /3/ on the basis of De Boer — Custers' model the method of theoretical calculation of the constant B and the affinity factor for organic matter adsorption from the gaseous phase by microporous sorbents was developed. According to the obtained relations

$$B = \frac{R^2\bar{2}6}{k^2\pi^2N^2K_0^2} \tag{6}$$

$$\beta = K/K_0 \tag{7}$$

where τ — the average radius of micropores, the value of which can be determined by the independent method of the low-angle x-ray dispersion; N — the number of the absorbent's atoms in 1 cm^3; K — Kirwood constant calculated through polarizability and magnetic susceptibility of the adsorbent and the adsorbate; k — the constant, dependent on the pore from (varies from 1.06 to 3/4 for different types of pores; in particular, for spherical pores k = 4/3);"O" index refers to the standard substance: Calculation of β for formula (7) takes into account not only the adsorbate properties but also the adsorbent's influence. In calculation of β (through molar volumes or parachors) the influence of the adsorbent's nature is taken into account by no way.

The obtained relationships refer to the adsorption of vapours. The further development of the given model conformably to the adsorption of organics from water solutions is based on the idea of the mean adsorption potential at adsorption of an organic molecule from the gaseous phase and the mean adsorption potential at adsorption of "n" molecules of water, displaced from the adsorption space by one molecule of adsorbate. One can show, that in this case

$$\beta = K_W/K_0 \tag{8}$$

where $K_W = K - n \cdot K_{H_2O}$ (9)

K — is the Kirkwood constant for the system "an organic matter — an adsorbent"
K_{H_2O} — is the Kirkwood constant for the system "water — an adsorbent"

Equations (6 - 9) reflect the relation between the equilibrium characteristics of organic substances adsorption from the gaseous phase and from water solutions with the structure and nature of microporous adsorbents. Thus it is principally possible to calculate isotherms of organics adsorption from gaseous and liquid phases on microporous adsorbents without carrying out the adsorption measurements.

Now let us turn to the kinetics of adsorption from solutions. The gradient of the sorbate's chemical potential is the moving force of the process of adsorption from solutions. The adsorption equilibrium condition is the equality of chemical potentials of the sorbable matter in the solution and in the adsorbent. Hence, the following differential Equation was suggested for description of the kinetics of adsorption from solutions /4/:

$$\partial\mu/\partial\tau = \alpha(\mu_p - \mu) \tag{10}$$

where α — the kinetic parameter independent on time; μ and μ_p are the running and equilibrium values of the chemical potential of the sorbate.

The solution of Equation 10 with the initial condition:

$$\tau = 0; \quad \mu = \mu_0 = \mu^0 + RT \ln \bar{\gamma} X_0 \tag{11}$$

(X_o — the molar fraction of a substance in the solution before adsorption; $\bar{\gamma}$ — the activity coefficient of the substance in water solution) and with the account of the thermodynamics of the maximum diluted solutions leads to the following integral Equation of the kinetics of the sorbate concentration change in the solution:

$$C = C_p \cdot \exp\left\{ \ln \frac{C_o}{C_p} \cdot \exp(-\alpha \bullet \tau) \right\} \qquad (12)$$

where the initial, running and equilibrium concentrations of the adsorbate in solutions are designated through C_o, C and C_p respectively. If we know the value of the running concentration C at some moment of time γ , from this Equation we can determine the kinetic parameter of adsorption under the static conditions expressed by α^*:

$$\alpha^* = \frac{1}{\tau} \cdot \ln \frac{\ln C_0/C_p}{\ln C/C_p} \qquad (13)$$

In order to obtain the correspondent integral equation of the adsorption kinetics for the constant and limited volume of solution (under so-called static conditions) it is sufficient to solve Equation 12 jointly with the known expression for Gibbs adsorption:

$$\Gamma = \frac{(C_0 - C) \cdot V}{m} \qquad (14)$$

where Γ —running adsorption;
V — volume of the solution;
m — the mass of the adsorbent.

Where $C = C_p$, $\Gamma = \Gamma_p$. In this case the dependence of the relative adsorption on the phases contact time will be expressed by the Equation:

$$\gamma^* = \frac{\Gamma}{\Gamma_p} = \frac{C_0 - C_p \exp\{\ln C_0/C_p \cdot \exp(-\alpha^* \tau)\}}{C_0 - C_p} \qquad (15)$$

If we mathematically direct V → ∞(or equivalently m → O) then we shall come to the case of such adsorption when the constant concentration of the sorbate is maintained around the adsorbent's grain at the interface (adsorption under the continuous-flow conditions). In order to obtain the specific expression of the adsorption kinetics equation for this particular case, it is necessary to turn to the limit in Equation 15 at $C_p \rightarrow C_o$:

$$\gamma = \lim_{C_p \to C_0} \gamma^* = 1 - \left\{ \exp \; -\alpha\tau \right\} \qquad (16)$$

In principle, one can determine the value of C_p, which is a part of Equation 15, by the grapho-analytical method at any given initial conditions of an experiment by joint solution of Equation 14 at $C = C_p$ and Equation of the adsorption isotherm 3 /4/.

One can strictly show that the kinetic parameters of adsorption under static (α^*) and dynamic (α) conditions are interrelated by the following Equation (for the same value of C_o):

$$\alpha = \alpha^* \frac{C_0 \Gamma_p \cdot \ln C_0/C_p}{(C_0 - C_p) \cdot \Gamma_0} \qquad (17)$$

This equation permits to calculate α using the determined under static conditions α^*, the isotherm of adsorption being know. In its turn this permits to make the calculation of an adworption filter with the stationary adsorbent's bed according to the suggested in paper /5/ mathematical model on the basis of the adsorption kinetics under static conditions. As compared to the method of obtaining breakthrough curves under dynamic conditions the latter one is rather attractive since the necessity of conducting labour-consuming experiments on the adsorption dynamics in columns with activated carbon falls away in this case.

In principle, the value of α may be calculated purely theoretically on the basis of the data on adsorption equilibrium. For technological and economic reasons it is desirable to carry out the adsorption treatment at a maximum possible possible rate, which can be reached at intra-particle diffusion as the limiting stage of the process. In practice, it can be realized at appropriate hydrodynamic conditions in the apparatus. In this case the kinetic parameter α may be presented as:

$$\alpha = \frac{6DF}{R^2} \qquad (18)$$

where D — the coefficient of the internal diffusion;
F — the coefficient of the geometric shape of adsorbent's grains;
R — the mean radius of the equivalent sphere.

The latter two values are the geometric characteristics of the adsorbent and can be determined independently. On the basic of the model of the intradiffusive transfer in micropores as in quasihomogenous adsorptive medium the internal diffusion coefficient may be presented as:

$$D = D_M P_\Sigma \frac{W_0}{W_\Sigma} \cdot \exp\left\{ -\frac{\bar{\epsilon}}{RT} \right\} \qquad (19)$$

where D_M — molecular coefficient of the substance diffusion in water calculated by Wilke and Chang formula;
P_Σ — the total internal porosity of the absorbent;
W_Σ — the total specific volume of the adsorbent's pores;
ϵ - f(θ) — the mean adsorption potential in sorbing pores corresponding to the degree of the

adsorption space filling θ at $C = C_o$. One can show that:

$$\bar{\epsilon} = \frac{\theta R \beta}{\sqrt{\pi B}\,\{1 - e2f\,\sqrt{\ln\theta^{-1}}} \qquad (20)$$

In Equations (19-20) R is the gas constant. Equations (17-20) express the relationship between the kinetic and equilibrium characteristics of the adsorption process and permit to theoretically calculate the adsorption kinetics both under static and dynamic conditions on the basis of the equilibrium data. They reflect the dependence of the kinetic parameters upon the structural characteristics of the adsorbent and upon the adsorbate nature.

For a number of reasons the exact solution of the adsorption dynamics problem obtained for some particular cases cannot be directly used for calculation of full-scale adsorption units. In connection with this one should note that it is not obligatory to know the whole breakthrough curve of adsorption dynamics. It is quite sufficient to be able to calculate the operating period of the filter before the breakthrough of impurities (γ_{np}) occurs, the period from the breakthrough to the total exhaustion of the sorbent's bed ($\Delta \gamma$) and the length of the mass-transfer zone (L_o) depending on various factors affecting the adsorption process under dynamic conditions, and in the first place, on such parameters as the height of the sorbent's bed in the filter (L), the specific filtering velocity (U), the initial concentration of the adsorbate in the feed solution (C_o), the size and the geometric shape of the sorbent's grains, porosity of the filter bed (ψ) etc. On the ground of the use of the adsorption dynamics stationary front and the described model of the kinetics of adsorption from solutions, the authors in work /5/ suggested the method of calculation of main dynamic parameters of the sorption filter operation, which may be presented by the following Equations:

$$\tau_{np} = \frac{L\,a_0}{U\,C_0} - \frac{2a_0\bar{R}\varphi}{UC_0\,(1-\varphi)} - \frac{\bar{R}^2\varphi}{DF\,(1-\varphi)} \qquad (21)$$

$$\Delta\tau = \frac{2\bar{R}a_0}{UC_0(1-\varphi)} + \frac{\bar{R}^2}{DF\,(1-\varphi)} \qquad (22)$$

$$L_0 = \frac{2\bar{R}}{1-\varphi} + \frac{\bar{R}^2 \cdot UC_0}{DFa_0\,(1-\varphi)} \qquad (23)$$

In these Equations a_o the adsorbate concentration in the solid phase, corresponding to the substance concentration C_o in the feed solution and expressed per unit of the adsorbent bed volume (in the same units as C_o). It is related to the correspondent value of the equilibrium adsorption expressed per one gram of the sorbent by the expression:

$$a_0 = \Gamma_0 \cdot d_H \qquad (24)$$

where d_H — bulk density of an adsorbent (g/l)

The given model is true when the adsorption isotherm is convex, $L \geqslant L_o$ and the adsorption kinetics is limited by the internal diffusion. The latter condition may be realized at $U > 1.8$ m/h, as was shown by a number of investigators. It should be noted that the adsorption treatment of natural and waste waters is carried on at higher filtering velocities, that is under such conditions when the adsorption process is surely limited by the intradiffusive transfer.

With the account of the above-stated it is clear that Equations 21-23 express the relation of dynamic parameters both with the conditions of the treatment process and the structural characteristics of the adsorbent and the nature of the adsorbate.

The mathematical model given in this paper can be assumed as the basis of the method of calculation of an adsorption unit for treatment of multi-component mixtures, in particular wastewaters. In this case the values of concentration and adsorption of organic impurities must be expressed in terms of the generalized index of wastewater pollution, for example by TOC and COD values.

Tahoe-Truckee Sanitation Agency Water Reclamation Plant

by

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The Tahoe-Truckee Sanitation Agency was formed on 1 May 1972 to carry out the mandate of the Porter Cologne Water Quality Act, a California law requiring exportation of all sewage from the Lake Tahoe Basin. Upon its formation, the Agency immediately embarked on a program to plan, program, design and construct a regional system which would transport all sewage from an area encompassing the California north shore of Lake Tahoe and Truckee to a regional plant. Construction of this new facility would result in the replacement of the existing interim treatment facilities operated by the Agency's five member entities: the Tahoe City Public Utility District, the North Tahoe Public Utility District, the Alpine Springs County Water District, the Squaw Valley County Water District and the Truckee Sanitary District. The Tahoe-Truckee Sanitation Agency (T-TSA) is governed by a Board of Directors comprised of five appointed directors, one from each member district.

Since it was mandatory that effluent be disposed of outside the Lake Tahoe Basin and in a manner that would protect the quality of any receiving waters and all potable water sources possibly affected by its disposal, the following alternative plans were identified by T-TSA as being worthy of study:

1) Exportation of treated wastewater to Long Valley, Sierraville or Loyalton for use as irrigation water on farmland.

2) Discharge of highly treated wastewaters to the Truckee River, either in Martis Valley or after the transportation of either treated or untreated wastewater to the Reno/Sparks treatment plant located in Nevada.

3) Land disposal of treated wastewater in the Truckee or Carson River Basins.

4) Upgrading existing treatment facilities in each of the member districts.

Retention of the effluent in the Truckee River Basin was decided to be of prime importance in order that the quality of water available to downstream users would not be diminished by this project. It was also determined that treatment of the wastewater to a high degree at a location near Truckee was the most economical solution and that this plan had the least adverse impact on the environment. Third, the existing high water quality standards required of the Truckee River could not be adhered to by upgrading of the existing treatment plants. The State of California's rigid waste discharge requirements necessitated the implementation of the most sophisticated treatment available

The project finally selected as the most advisable required the construction of an interceptor line from Tahoe City to Truckee, California, the construction of a 4.83 MGD regional sewage treatment plant in Martis Valley, and the installation of an underground disposal system that would allow the effluent to percolate into the permeable glacial outwash soil near the plant. Full tertiary treatment, including maximum removals of nitrogen and phosphorus, was deemed necessary to protect the quality of the Truckee River into which the effluent would ultimately find its way and to ensure the safety and integrity of this primary water source for Reno, Nevada, a major city downstream.

Studies of the background concentration of pollutants in the Truckee River revealed that the critical pollutants were total nitrogen and total phosphorus. These were determined to be the most difficult standards to meet; other pollutants could easily be removed by proven processes. The primary goal of process design was to ensure that the treated effluent, when mixed with background concentrations existing in the river, would not violate the river standards even if discharged directly into the river. Percolation into the Martis Valley soils would provide an additional treatment factor wherein a residence time in the soil of about 150 days is experienced. The effluents's mixing with surface water percolating into the soil would result in additional dilution of its pollutants.

T-TSA's regional facilities became operational in February 1978. The reclamation plant was designed to produce the effluent quality shown in Table I. Treatment processes selected for the project are shown in Table II.

Figure 1 depicts the process flow diagram illustrating the general arrangement of these processes.

Treatment Processes

After thorough review of the requirements for effluent quality, it was concluded that only a combination of biological and physical-chemical treatment could meet the applicable criteria.

Primary Treatment

Primary treatment removes a large portion of the settleable organic and inorganic solids entering the plant. Primary treatment facilities consist of the headworks, grit removal chamber, primary clarifiers and primary sludge pump station.

Headworks

Raw sewage enters the headworks flow distribution channel and normally passes through one of two comminutors. The comminutors shred the coarse solids in the sewage into smaller particles. This reduces the possibility of large organic solids settling in the grit removal system or continuing through the plant and clogging pumps and sludge lines.

Grit Removal

The grit removal system removes heavy inorganic solid materials such as sand and gravel, thus protecting moving mechanical equipment within the plant from abrasive wear and preventing grit accumulation in basins and digesters.

The comminuted sewage is distributed across the entire grit chamber so that the velocities are reduced to about one foot per second which allows the heavy materials to settle and be removed.

Settled grit is pumped to the grit cyclone where most of the inorganics are separated from the lighter organics. The grit washer removes the remaining organics, and the grit is discharged to a bin for disposal.

Primary Clarifiers

The primary clarifiers provide relatively quiescent conditions which allow settleable solids to be removed, and thus reduce the organic loading on the subsequent treatment processes. Supernatant from the clarifiers flows to the biological treatment process. The settled solids (primary sludge) are pumped to the digester by pumps located in the primary sludge pump station.

Table 1

Design Criteria — Effluent Quality

5-day BOD	2 mg/l or less
COD	10 mg/l or less
Suspended solids	1 mg/l or less
MBAS	.1 mg/l or less
Turbidity	1 JTU or less
Total nitrogen	2 mg/l or less
Total Phosphorus	.1 mg/l or less
Coliform organisms	less than 2.2 mpn/100 ml

Table 2

- Primary Treatment
  - Comminution
  - Flow metering
  - Grit removal
  - Primary sedimentation (clarification)
- Biological Treatment
  - Pure oxygen activated sludge
- Chemical Treatment
  - Rapid mix and flocculation with lime
  - Chemical clarification
  - First stage recarbonation
  - Recarbonation clarification
  - Second stage recarbonation
- Advanced Waste Treatment
  - Dual media filters
  - Activated carbon adsorption
  - Ammonia removal using clinoptilolite (ion exchange)
- Disinfectin Using Chlorine
- Solids Handling and Side Stream Systems
  - Anaerobic digestion of organic sludge
  - Filter press dewatering of chemical and digested sludges
  - Land disposal of sludge
  - Ammonia removal and recovery from regenerant
  - Activated carbon regeneration

Biological Treatment

Biological treatment removes most of the soluble and finely suspended organic matter in sewage. The biological treatment system utilizes a pure oxygen-activated sludge process and consists of oxygenation basins, carbon dioxide stripping, secondary clarifiers, and the return and waste activated sludge pump station.

Oxygenation Basins

Primary effluent is mixed with return activated sludge and some plant return flows in a common mixing

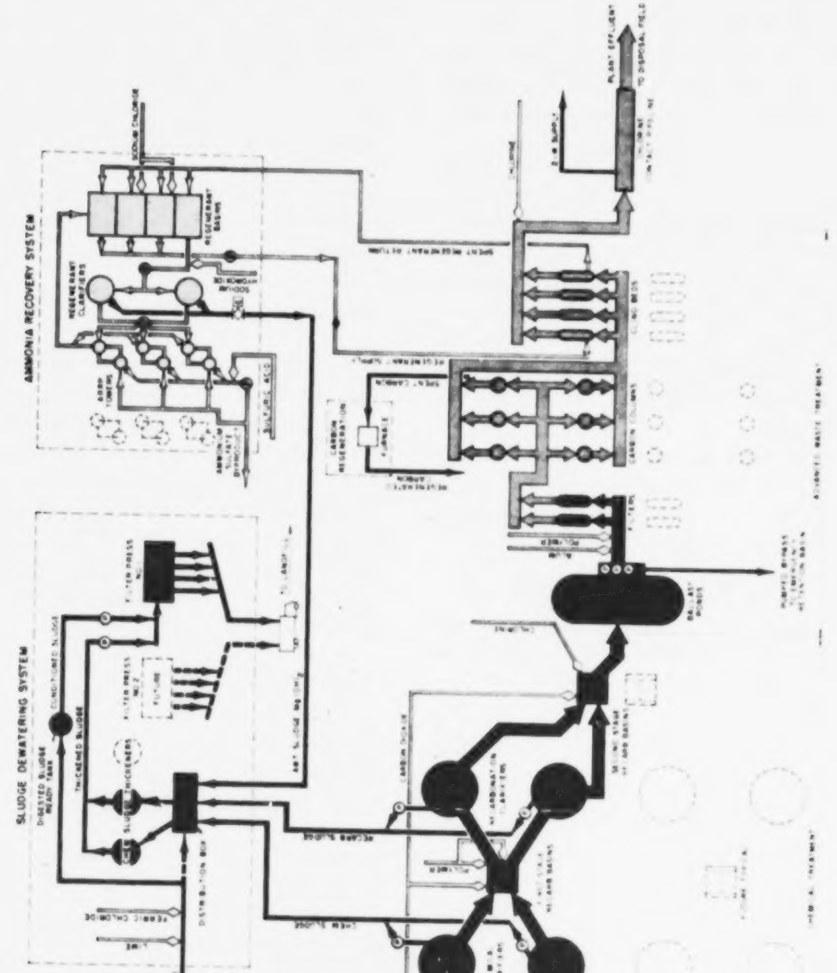
GRIT DISPOSAL
ORGANIC SLUDGE DIGESTION
SLUDGE DEWATERING SYSTEM
AMMONIA RECOVERY SYSTEM
DISTRIBUTION BOX
PRIMARY DIGESTER
SECONDARY DIGESTER
SECONDARY CLARIFIERS
CHEMICAL CLARIFIERS
RECARBONATION CLARIFIERS
BALLAST PONDS
FILTERS
CARBON COLUMNS
CARBON REGENERATION
STRIP TOWERS
REGENERANT CLARIFIERS
PLANT EFFLUENT
TO DISPOSAL FIELD

Figure 1
Plant Process Diagram

chamber prior to entering the oxygenation basins. The flow then enters the flow distribution channel where it passes through submerged gates to the first stage of the oxygenation basin trains in service. Actual flow division between basins is determined by overflow weirs at the outlet of the basins.

Each oxygenation basin train consists of three separate covered stages in series. Pure oxygen is supplied at a controlled rate to the first stage. Each stage is equipped with a mechanical aerator for transferring the oxygen to the liquid and for keeping the basin contents completely mixed. The liquid (mixed liquor) and gas flow through ports to the second stage. The gas space is vented to the atmosphere at a controlled rate at the end of the second stage, and the mixed liquor flows to the third stage through a submerged port.

The third stage is used to strip carbon dioxide gas from the mixed liquor and gas space, in addition to providing additional biological treatment to the wastewater. One end of the third stage is vented to the atmosphere while a CO_2 compressor pulls gas from the other end, stripping CO_2 with the air flow. The stripped air containing a relatively high concentration of carbon dioxide (± 3 percent) is drawn from the third stage, compressed, and used in the recarbonation basins along with the combustion gases from the boilers.

Secondary Clarifiers

The secondary clarifiers provide a quiescent condition that permits the biological solids in the mixed liquor to settle and form a sludge. The clarified wastewater, now low in organics, overflows from the secondary clarifiers to the chemical treatment process. Settled sludge is removed from the secondary clarifier through sludge suction lines that are attached to the lower rotating clarifier mechanism.

Oxygen Generation

High purity oxygen is generated by an on-site PSA (pressure swing adsorption) generator. Compressed air is fed to the generator, and it separates the nitrogen and other impurities from the air producing a relatively high purity oxygen. A backup liquid oxygen (Driox) storage tank and vaporizer provide oxygen to the oxygenation basin when the PSA generator is out of service or cannot meet the demand.

Chemical Treatment

In the chemical treatment process, lime is added to secondary effluent to precipitate phosphorus as calcium hydroxyapatite, $Ca_{10}(OH)_2(PO_4)_6$, which is then removed by settling in the chemical clarifier. In addition to forming calcium hydroxyapatite, the lime also precipitates calcium carbonate, $CaCO_3$, and magnesium hydroxide, $MgOH_2$, which is similar to the lime softening process in water treatment. Although the primary purpose of the lime treatment process is to remove phosphorus, it also reduces the suspended solids which are carried over from the secondary clarifiers by trapping the solids in the chemical precipitates (floc). Since these organic solids contain C.O.D. and viruses, removal of these impurities is enhanced.

The chemical treatment system consists of the rapid mix basins, flocculation basins, chemical clarifiers and chemical sludge pump station.

Rapid Mix Basins

Secondary effluent enters the rapid mix basin inlet chamber along with chemical sludge thickener overflow and chemical sludge recycle (if practiced). Lime is added at the inlet to the rapid mix basins to raise the pH to about 11.2. Each of the rapid mix basins contains a mechanical mixer which rapidly disperses the lime slurry throughout the secondary effluent. Secondary effluent can be bypassed to the emergency storage pond from the rapid mix inlet chamber if necessary because of a process failure.

Flocculation Basins

The high pH wastewater flows from the rapid mix basins into parallel flocculation basins. The purpose of the flocculation basins is to gently mix and agglomerate the coagulated precipitates into large floc particles for better settling in the chemical clarifiers.

Each flocculation basin is equipped with two vertical adjustable-speed paddle mixers. Polymer can be added at the influent and effluent end of both basins as an aid to flocculation and settling.

Chemical Clarifiers

Flocculated wastewater flows from the flocculation basins to two chemical clarifiers where the solids are allowed to settle. Overflow from the clarifiers, which is now low in phosphorus and at a high pH, moves on to the recarbonation process. Chemical solids that settle to the bottom of the clarifiers are scraped to a sludge sump by the clarifier scraper mechanism.

Chemical Feeding

Lime is stored in two bins and is fed to two detention-type slakers. Lime slurry is continuously pumped from the slurry tank to a pipe loop around the plant and back to the slurry tank in order to keep the lime is suspension. A slurry is prepared and fed to a lime slurry receiving tank on demand. Lime slurry is bled off the pipe loop to the rapid mix basins where it is injected and controlled by a pH feed back control loop.

Polymer solution, prepared automatically from dry polymer, is pumped to a distribution panel in the chemical sludge pump station where it can be distributed to feed points at the flocculation basins and first-stage recarbonation.

Recarbonation

The primary purpose of the recarbonation system is to inject carbon dioxide (CO_2) gas into the wastewater to lower the high pH resulting from lime treatment. The two-stage system with intermediate settling also provides for maximum removal of calcium carbonate which reduces the calcium and total dissolved solids (TDS) content of the water below that which would result from the same pH reduction in a single stage. The intermediate settling step also provides an additional minor reduction of phosphorus.

The three sources of carbon dioxide for the recarbonation system are (1) compressed stack gas from the boilers in the digester building, (2) stripped gas from the oxygenation basins, and (3) an auxiliary 31-ton liquid carbon dioxide storage facility that supplements the first two CO_2 sources.

First Stage Recarbonation

The pH is reduced to approximately 10 in the first stage recarbonation basin. This pH reduction occurs as a result of the reaction of the carbon dioxide with the hydroxyl alkalinity remaining from the lime treatment. This reaction causes carbonates to be formed which react with calcium ions forming a settleable precipitate, calcium carbonate.

Effluent from the chemical clarifiers enters the first-stage recarbonation basins through slide gates at the inlet distribution box. Carbon dioxide is injected through distribution headers in the basins and is controlled by motorized valves positioned in response to pH control loops. Polymer can also be added at the discharge end of these basins to aid in conditioning the sludge for settling.

Clarification

Effluent from first-stage recarbonation flows to the recarbonation clarifiers. Calcium carbonate sludge settles to the bottom of the clarifiers and is mechanically scraped to the center hoppers and removed by the recarbonation sludge pumps located in the chemical sludge pump station. Sludge is pumped to the chemical sludge thickener with a portion recycled to the distribution box of the first-stage recarbonation basins. The recarbonation clarifiers are identical to the chemical clarifiers.

Second-Stage Recarbonation

The pH is reduced to the normal range of 7 to 7.5 in second-stage recarbonation. Carbon dioxide is automatically applied to distribution headers in the basins through motorized valves positioned in response to pH control loops. The recarbonated effluent flows by gravity to the ballast ponds.

Filtration

It is the purpose of the filtration system to provide the maximum water clarity achievable for several reasons: (1) the discharge requirements specify that the mean effluent turbidity shall not exceed 2 NTU; (2) filtration ahead of carbon adsorption reduces the load of organics, suspended solids, and colloidal matter reaching the carbon which increases carbon efficiency and reduces carbon fouling; and (3) the effectiveness of disinfection by chlorination is significantly improved by removing particulate matter containing entrapped viruses, bacteria, and other chlorine-demanding materials.

Activated Carbon Adsorption

The filtered effluent flows directly to the granular activated carbon columns. In biological treatment of wastewater, dissolved organic materials are removed including most of those measured by the BOD (biochemical oxygen demand) test. However, biological processes do not remove all of the undesirable organic materials, as measured by the COD (chemical oxygen demand) test. These materials include MBAS (methylene blue active substances—primarily from detergents), herbicides, pesticides, tannins, lignins, ethers, proteinaceous substances, and other color and odor producing organics. It is the purpose of the activated carbon adsorption process to remove the organic materials which are not adequately removed in the biological process. Activated carbon adsorption also has the ability to provide some removal of trace inorganics such as cadmium, chromium and silver. Activated carbon is especially effective in eliminating taste, odor, and color as a result of the above-described removal of organic materials.

Removal of soluble organics is important for several reasons: (1) the discharge requirements specify that the mean COD shall not exceed 15 mg/l, and the mean MBAS shall not exceed 0.15 mg/l; (2) the organic content of the reclaimed water should be minimized to eliminate taste, odor and foam; and (3) the removal of organics reduces the chlorine demand of the wastewater, reducing the cost for effective disinfection.

Ammonia Removal

The carbon treated effluent flows to the clinoptilolite ion exchange beds for ammonia removal. Clinoptilolite is a naturally occurring mineral (zeolite) found in such areas as Hector, California and Yuma, Arizona. The mineral is mined and then crushed to a 20 × 50 mesh size (similar to fine sand). Consequently, the clino beds located at the end of the waste treatment process serve as polishing filters and remove carbon fines which would otherwise pass out with the effluent reducing effluent suspended solids and COD.

Four horizontal pressurized clino beds operate in parallel. These beds are very similar to the filters. Normally, three beds are on line and the fourth is undergoing regeneration. The ion exchange beds operate in two basic cycles: (1) service cycle which captures the ammonium ions on the beds, and (2) regeneration cycle which elutriates the captured ammonium ions from the beds to a regenerant solution. During the service cycle, the beds are "on line" and are part of the treatment process. During regeneration, the beds are completely disassociated from the main treatment process and are part of the ion exchange bed regeneration system (described in a following section). During the service cycle, flow passes downflow through the beds. In regenerating, the flow is reversed to an upflow direction.

All of the flow passing through the carbon columns must pass through the clino beds during the service cycle. The rate of flow or flow balance through the clino beds is manually adjusted, if necessary, by the clino bed effluent valves. Flow indication is provided at each clino bed for balancing or adjusting flow rates.

Service cycle duration is determined by the number of bed volumes treated and the ammonium ion concentration. The service cycle is considered to end when the ammonia-nitrogen concentration in the effluent reaches a "breakthrough" concentration of 2 mg/l.

Chlorination

Chlorine may be applied to the wastewater at four points in the treatment process. These points are (1) plant headworks, (2) effluent from the oxygenation basin (mixed liquor), (3) C&CT effluent (prior to the ballast ponds), and (4) final AWT effluent.

Chlorination at the headworks is a standby process for odor control. Normal odor control is achieved by the addition of hydrogen peroxide.

A chlorine feed point is provided to the oxygenation basin mixed liquor lines to help control filamentous growths and nitrification if needed.

Chlorination of the C&CT effluent is for control of growths in the ballast ponds and filters and is used only if necessary.

AWT effluent chlorination provides final disinfection and reduces ammonia nitrogen concentration if necessary to comply with waste discharge requirements. Chlorine solution is injected into the 30-inch pipeline to the disposal fields. Theoretical detention time at design flow is 15 minutes.

There is one continuous chlorine residual analyzer. A residual sample is taken from the plant effluent line approximately 30 seconds from the point of chlorine injection. The primary chlorinator receives a dosage signal from this residual analyzer which also varies the chlorine feed rate to maintain the set point residual.

Effluent Disposal Field

The plant effluent, still under pressure, flows through a 30-inch pipeline to eight disposal fields located about 1,500 feet south of the treatment plant. These eight subsurface disposal fields are sized for an initial design flow of 4.83 MGD under normal operation which includes at least a 50-percent "resting" cycle. Treatment plant operators determine the number of fields in operation by referring to an application rate chart, manually selecting the fields for operation, and setting a time which alternates the fields from service to rest mode. Fields are normally alternated every 12 hours.

Organic Sludge Thickening

Waste activated sludge is pumped to the organic sludge thickener for thickening prior to being fed to the digesters. This allows the digesters to be operated with longer hydraulic detention times which produces a more stable digested sludge than would be experienced with the less concentrated waste activated sludge.

The sludge will normally thicken to a 4 to 5 percent solids. Thickened sludge is pumped from the thickener to the digesters by two thickened WAS pumps. Supernatant overflows the thickener and discharges to the oxygenation basin.

Organic Sludge Digestion

Organic sludge digestion reduces the total mass of volatile sludge by over 55% with most of the degraded organic material being converted to methane and carbon dioxide gas. The digesters handle organic sludges from two major sources: (1) the settleable solids contained in the raw sewage (primary sludge), and (2) the waste activated sludge from the oxygen-activated sludge process.

Thickened organic sludge enters the primary digesters through eductors in the sludge recirculation lines. The organic sludge includes thickened WAS, primary sludge and primary scum. Each of the primary digesters is completely independent with a sludge recirculation system, heat exchanger and gas mixing system. Normally one primary digester will be maintained in an empty-standby mode to be used for storage in the event of a filter press outage.

Sludge flows by gravity from the primary digesters to the secondary digester where the solids settle and thicken, along with some additional solids breakdown. The secondary digester has a floating cover for gas storage. The thickened sludge is drawn off the bottom of the secondary digester and pumped to sludge dewatering facilities, and the supernatant is drawn off and returned to the plant headworks. Digester gas is used to fire boilers—the flue gases of which are rich in CO_2 and are used in the recarbonation process.

Chemical Sludge Conditioning

Chemical sludges are thickened by gravity settling to remove excess liquid prior to dewatering. The thickeners also provide storage of chemical sludges.

Chemical sludge is produced at three points in the treatment process: chemical clarifiers, recarbonation clarifiers and regenerant clarifiers that are part of the regenerant recovery system in the AWT building. These sludges are pumped to the chemical sludge distribution box and divided between the two thickeners. The sludge will normally thicken to 10-12 percent solids. Thickened sludge is drawn from the thickeners by the filter press feed pumps.

Dewatering

A filter press system is used to dewater digested organic and chemical sludge prior to land disposal. Digested sludge is dewatered to about 45 percent solids. Chemical sludge is dewatered to about 55 percent solids.

Digested sludge is drawn from the ready tank and pumped to the filter press for dewatering by selected filter feed pumps. At design flow, approximately 5 hours of filter press operation daily is required for dewatering digested sludge. Chemical sludge is drawn from both chemical sludge thickeners by any filter feed pump. Approximately three hours of filter press operation daily is required for dewatering chemical sludge at design flow.

The filter press consists of eighty 64-inch by 1.18 inch thick chambers. Other pieces of equipment in the system include: (1) a filtrate storage tank, (2) a filtrate measuring weir, (3) a control panel, and (4) an acid storage and wash tank.

The filter press system operates semiautomatically. The control panel features allow for control of the complete dewatering cycle in accordance with four subcycles:

Subcycle 1 (organic sludge supply cycle) — low ready tank level starts the sludge transfer pumping.

Subcycle 2 (organic sludge conditioning) — the chemical feeders and reaction tank mixer start when the sludge transfer pumps are started.

Subcycle 3 (filtration) — the filtration cycle is automatic but is initiated manually by the filter press operator.

Subcycle 4 (discharge) — filtration cycle is stopped automatically and the operator initiates the discharge of the cakes.

Upon manual initiation by the operator, all sludge cakes are discharged and the unit is ready for another cycle.

Filtrate from the dewatering process flows through a weir box and into a filtrate tank, and then returns to the rapid mix or oxygenation basins. The flow is measured. When it reaches a preset low flow, the filtration cycle is stopped and the operator is alerted.

Calcium carbonate buildup in the filter press must be removed periodically by washing with dilute acid. Muriatic acid from the acid storage tank is diluted and pumped through the filter press. The acid solution is returned to the dilute acid tank for reuse.

Activated Carbon Regeneration

Granular activated carbon removes detergents, insecticides, herbicides and various organic substances which contribute to the taste, odor and color of the wastewater. The carbon increases in weight and with continued use eventually becomes saturated and loses its ability to further adsorb organic materials. The carbon nearest to the inlet (bottom) of the carbon column becomes saturated or exhausted first. As this occurs, the exhausted carbon is removed for regeneration and is regenerated. Previously regenerated or fresh carbon is added to the top of the column to return the carbon bed to its original depth.

The regeneration process restores the adsorptive capacity of the granular carbon. Regeneration is accomplished by heating the carbon to temperatures in excess of 1,700°F. The heat vaporizes, drives off and oxidizes the impurities which have been adsorbed on the carbon—ultimately restoring the carbon very nearly to its original activity.

As the carbon in the bottom of a carbon column becomes saturated with impurities and loses its capacity to adsorb organics from the incoming wastewater, approximately 10 percent of the carbon in one column is withdrawn and discharged to a dewatering bin. Carbon is transferred from one container to another in slurry form. About one gallon of water per pound of carbon is required to form a suitable slurry.

The dewatering bins are epoxy-coated to protect the steel against the corrosive action of partially dewatered carbon. It requires about 10 minutes for the free water to drain from the carbon through the screened drains. Because of the fine pore structure of the carbon, it still retains 40-45 percent moisture after draining, which is about optimum moisture content for thermal regeneration. The partially dewatered carbon is transferred from one of the two dewatering tanks to the carbon regeneration furnace inlet at the top of the furnace by means of a variable speed, stainless steel screw conveyor at rates which can be varied from 80-500 pounds per hour (dry basis).

The carbon regeneration furnace is a gas-fired, six-hearth unit rated at 3,825 pounds per day of dry carbon. There are six burners, two each on Hearths 4, 5 and 6. Temperatures on these hearths are independently controlled within 10°F by an automatic temperature controller. All hearth temperatures are recorded. Proportional flowmeters on the burner air-gas mixture lines ensure a constant percentage of excess oxygen. The oxygen content of the furnace atmosphere must be limited to avoid overburning of the carbon. Each burner may be adjusted to maintain 0.5 percent oxygen by volume. In addition, steam can be added to Hearths 4 and 6 to give more uniform distribution of temperatures throughout the furnace.

The carbon is moved across the hearths by four rotating stainless steel rabble teeth which are oriented vertically. The rabble arms are driven by a central shaft. Cooling air is blown through the hollow rabble arms and central drive column shaft. The exhausted cooling air is ducted into the main exhaust stack. Furnace gases pass from the top of the furnace through an afterburner and enter the precooler and scrubber which eliminates odors and dust from the exhaust.

The regenerated carbon is discharged from the bottom of the furnace into a quench tank for cooling. There are several pressure water jets strategically located to keep the carbon moving in the quench tank and pump suction lines. The carbon slurry is pumped from the quench tank by eductors to either of two transfer and de-fining tanks. The carbon is washed in the de-fining tanks to remove fines and then is ready for transfer back to the columns for reuse.

Clinoptilolite Bed Regeneration

Clinoptilolite bed regeneration removes the ammonium ions from the exchange sites on the clinoptilolite media. With periodic regeneration, the clino beds will effectively and continuously remove ammonia nitrogen from the wastewater. The ammonium ions are removed and recovered from the spent regenerant so that the regenerant solution can be reused.

When the clino bed effluent ammonia-nitrogen concentration exceeds 2.0 mg/l, breakthrough has occurred, and the bed must be regenerated. The regeneration cycle is initiated manually or automatically. Automatic cycle initiation is normally determined by the total flow passing through the bed since the last regeneration cycle. The regeneration cycle involves over 30 completely automated steps which must take place in the proper sequence. Basically, the following steps occur during a regeneration cycle: (1) surface wash, (2) backwash, (3) purge liquid bed content to waste with regenerant, (4) ammonion elutriation cycle, (5) purge regenerant solution to basins with influent, (6) rinse bed to waste, and (7) return bed to "on line" position or service cycle. The pressure vessels which contain the clino beds are divided into two compartments to reduce backwash flow requirements. Only one-half of the vessel is backwashed at a time.

During elutriation, regenerant solution is passed upflow through the bed in four phases. In each phase, 10 bed volumes of regenerant solution are passed through the bed. The four phases of regeneration are staged so as to initially produce a spent regenerant solution with a high ammonia concentration for later recovery processes, and save the highest quality regenerant solution for the last 10 bed volumes which ensures maximum regeneration of the clino media. By concentrating the ammonia-nitrogenin the spent regenerant during the early phases of elutriation. The total amount of regenerant solution passing through the regenerant recovery system is significantly reduced.

The purge steps reduce the loss and/or dilution of regenerant solution. The surface wash, backwash and rinse steps serve the same function as for the filters.

Regenerant Clarification Process

Spent regenerant from elutriation is pumped from the basin with the highest ammonia concentration, basin 4, to the constant head box. Sodium hydroxide is added as the spent regenerant leaves the constant head box to raise the pH to approximately 11.4 in order to convert the ammonium ions to ammonia gas.

Magnesium ions are also removed by the ion exchange bed and are therefore present in the regenerant. The regenerant solution must be clarified to remove the magnesium hydroxide precipitate so that subsequent clogging of the clino media does not occur.

Flow from the constant head box is split equally to two regenerant clarifiers. Magnesium hydroxide sludge settles in the clarifiers, is allowed to thicken as much as possible and is periodically blown down to the chemical sludge thickener automatically. High pH clarified spent regenerant flows from the clarifiers to the stripper supply pumped from the wet well. The regenerant is pumped from the wet well to the ARRP modules.

Ammonia Removal and Recovery Process

The Ammonia Removal and Recovery Process (ARRP) is a closed-air-cycle system which strips dissolved ammonia gas from the spent regenerant and reabsorbs the ammonia gas in a sulfuric acid solution. Spent regenerant passes vertically down through the stripping tower while a countercurrent closed loop air stream flows upward. The ammonia-gas-laden airstream leaves the top of the stripping tower and enters the bottom of the absorber tower where the acidic absorber solution scrubs the ammonia gas from the air stream, which returns to the bottom of the stripping tower for the next pass through that tower.

The absorbed ammonia gas combines with sulfuric acid in the absorber solution to form ammonium sulfate, a common chemical fertilizer. A portion of this solution is automatically pumped to the ammonium sulfate storage tank when the ammonium sulfate reaches a strength of about 40 percent. As the ammonium sulfate is being blown down to the storage tank, 50 percent strength sodium hydroxide is added to adjust the solution to near neutral pH.

Spent regenerant, after passing through the ARRP, is then capable of being reused for elutriating ammonium ions from exhausted clino beds. The recovered regenerant that passes through the ARRP is low in ammonia, approximately 30 mg/l, and is discharged into Regenerant Storage Basin 1. This represents the highest quality regenerant solution in the basins and is the regenerant solution pumped through the clino beds during the last 10-bed volumes of that 40-bed volume regeneration cycle. Sodium chloride brine is added to the regenerant solution to make up for salt losses which occur during the purge cycles and magnesium hydroxide sludge blowdown.

Energy Consumption

During the evaluation of the various treatment processes, much consideration was given to the efficient use of energy and to the possible recycling of various constituents in the sewage.

In the case of the ammonia removal and recovery system, ammonium sulfate is produced and is periodically removed from the plant by truck for use as a fertilizer. It is also anticipated that a program will be developed for use of the chemical sludge produced in the lime treatment step on agricultural lands, although agricultural lands are a considerable distance from the plant. In this manner, the calcium carbonate and calcium phosphate compounds which can have value to certain agricultural soils will be returned to the land in an environmentally aesthetic program.

The plant utilizes anaerobic digestion for the accumulated organic solids. The methane gas byproduct of this process is used for plant heating and may eventually be used for reducing the plant electrical load requirement. Carbon dioxide produced from the burning of the methane gas is used for the recarbonation processes following lime treatment. The stabilized organic solids will also be returned to agricultural lands if a market can be found.

Comparisons were made of energy consumption for the following processes:

Oxygen vs. air for biological treatment;

Ozone vs. chlorine for disinfection;

Chemical vs. electrolytic processes for the ammonia stripping-recovery process; and

Incineration vs. land disposal for both organic and chemical sludge.

On-site incineration and calcining were eliminated from the plant as a result of these energy analyses.

Perhaps the most interesting of the various processes used in the plant is the use of clinoptilolite, a natural zeolite, and its regenerant recovery. Before the selective ion exchange process was chosen, detailed consideration was given to three other possible treatment processes for nitrogen removal: biological nitrogen removal, breakpoint chlorination, and ammonia stripping.

Considering the Truckee area climate, where winter temperatures have dropped as low as -40°F, and the anticipated loading variations, the ion exchange process using clinoptilolite appeared to be the only process which could reliably meet the 2 mg/l total nitrogen discharge standard. The key to the use of the ion exchange process lies in the processing necessary for recovery of the spent regenerant solution. The process is generally of little value unless this solution can be recovered by removal of the accumulated ammonium ions. Coincident with the nitrogen removal study, the firm of CH2M Hill, of Redding, California, designers of the T-TSA facility, were in the process of the development of an ammonia removal and recovery process (ARRP) which appeared to fulfill this need. Pilot studies subsequently confirmed that this new process complemented the ion exchange process and offered numerous advantages over the other three alternatives which had been previously considered for recovery of the regenerant solution: air stripping, electrolytic cells and steam stripping. Air stripping had the significant disadvantages of freezing and low temperature problems, calcium carbonate scaling and discharge of ammonia into the atmosphere. Electrolytic cells used to accomplish ammonia removal by breakpoint chlorination had the significant disadvantages of high electrical energy demand, high operating costs and potential scaling of the electrodes. The steam stripping process also had three major disadvantages: high energy demand, use of high pH regenerant and associated media clogging and attrition problems, and recovery of only a very dilute ammonia fertilizer.

The new ARRP process appears to overcome these disadvantages. Advantages include:

1) No freezing or air temperature limitations.

2) Scaling reduced to a minor maintenance item.

3) No discharge of ammonia or other waste product into the atmosphere.

4) Low energy consumption—equivalent to normal fertilizer production.

5) Recovery of a valuable, concentrated, conventional chemical fertilizer—ammonium sulfate.

6) Use of a 9-11 pH regenerant with no media fouling or measurable attrition.

Although the process requires the addition of caustic soda (NaOH), and some makeup salt (NaCl), the offsite energy and environmental effects were also considered to be a positive factor. Caustic production requires considerable offsite electrical consumption, but this offsite energy consumption together with the internal plant energy consumption attributable to ammonia removal is still less than the amount necessary for equivalent nitrogen fertilizer production using normal industry methods. The process is also much less energy intensive than other nitrogen removal processes.

Another rather unusual and interesting feature of the T-TSA water reclamation plant is that even though the plant effluent is of very high quality, with virtually all contaminants removed, current regulations still prohibit it from being directly discharged into the receiving waters of the Truckee River Basin. This requirement came about because of the pristine quality of the Truckee River in this vicinity and because it is the primary source of the Reno, Nevada water supply. Consequently, as a further precautionary measure, the effluent is discharged into an underground disposal system near the plant.

The combination of advanced waste treatment processes and the underground disposal system is believed to provide the best assurance possible of the protection and preservation of this environmentally sensitive and pristine receiving water.

Another unique feature in this treatment facility is the use of a combination piping, electrical conduit, pedestrian and light-vehicle corridor which runs through the center of the plant. This corridor was deemed necessary to ease the operators' burden of maintaining and operating the numerous chemical feed lines during extreme weather conditions. It also allows flexibility in the areas of additions of future facilities. Two-way electrical light vehicles facilitate the operators' movement through the corridor from one end of the plant to the other, a distance of nearly one half mile.

Architecturally, the plant was designed with the idea of low maintenance, as first priority, coupled with an environmentaly attractive appearance. Architectural features include a combination of split face masonry block, Corr-ten (self-rusting steel) on an extended manzard frame, and translucent panels in the larger buildings.

As would be expected, an advance waste treatment facility utilizing all of the previously described processes and located in an extremely varied climate is expensive. The construction bid for the plant was $19.2 million, and the total project cost including the effluent disposal system, a regional interceptor sewer, and other indirect costs amounted to about $32 million. Current sewer use charges for maintenance and operation of the regional facilities for each dwelling unit are $7.50 a month, which is minimal considering the high degree of wastewater treatment and the protection afforded the surrounding environment.

Startup and Operation of the Tahoe-Truckee Sanitation Agency Advanced Wastewater Treatment Plant

Thomas J. Kennedy, Plant Manager
Ossian R. Butterfield,
General Manager/Chief Engineer
Craig F. Woods, Operations Engineer

Introduction

The Tahoe-Truckee Sanitation Agency began operation of its 18,300 cu m/day (4.83 MGD) advanced waste treatment facility on February 1, 1978, and has completed the startup phase of plant operation. This report details operations, problems, and plant performance during startup. Because of the plant's overall complexity a thorough discussion of its design features and design parameters is beyond the scope of this presentation.

Briefly, the major sewage treatment processes employed are as follows: Primary clarification; pure oxygen activated sludge (Pressure Swing Adsorption); lime treatment and chemical clarification for phosphorus removal; two stage recarbonation with intermediate calcium carbonate settling; mixed media filtration; activated carbon adsorption; ion exchange ammonia removal using clinoptilolite; and disinfection. Sidestream processes include sludge thickening; anaerobic digestion of organic sludges; sludge dewatering using high pressure filtration of chemical and organic sludges; thermal regeneration of activated carbon; clinoptilolite regenerant recovery using chemical clarification and ammonia stripping. The process flow diagram is presented in Figure 1.

Sewage originates from the communities on the North and West Shores of Lake Tahoe and along the Truckee River Corridor from Lake Tahoe to Truckee, California. The area's major industry is tourism which has two primary seasons: the winter ski season and the summer season. Heavy influx of tourists results in extremes of sewage flows and loadings over holidays, weekends and seasonal peaks.

Effluent from the plant is indirectly discharged to the pristine receiving waters of the Truckee River and consequently, the discharge limits (Table 1.) imposed upon the Agency by the State Water Resources Control Board are stringent, necessitating a very high degree of treatment.

Historical

The Tahoe-Truckee Sanitation Agency was formed on 1 May 1972 to carry out the mandate of the Porter Cologne Water Quality Control Act, a California law requiring the exportation of all sewage from the Lake Tahoe Basin. The newly formed Agency's initial task was to consider the design and construction of a regional wastewater treatment facility that would result in the replacement of existing interim facilities operated by its member entities: the Tahoe City Public Utility District, the North Tahoe Public Utility District (both located in the Lake Tahoe drainage basin), the Alpine Springs County Water District, the Squaw Valley County Water District and the Truckee Sanitary District which are located in the Truckee River drainage basin (Figure 2.). The Agency is governed by a Board of Directors comprised of five appointed directors, one from each member district.

Retention of the effluent in tne Truckee River Basin was decided to be of primary importance in order that the quantity of water available to downstream users not be diminished by this project. It was also determined that treatment of the wastewater to a high degree at a plant near Truckee was the most economical solution and that this plan had the least adverse impact on the environment. Thirdly, the existing high water quality standards required of the Truckee River could not be adhered to by simply upgrading the existing treatment plants. The State of California's rigid waste discharge requirements necessitated the implementation of the most highly sophisticated treatment available.

The project which was finally selected from many alternatives studied required the construction of an interceptor line from Tahoe City to Truckee, California, the construction of a 4.83 MGD advanced waste treatment plant in Martis Valley near Truckee, California, and the installation of an underground effluent disposal system

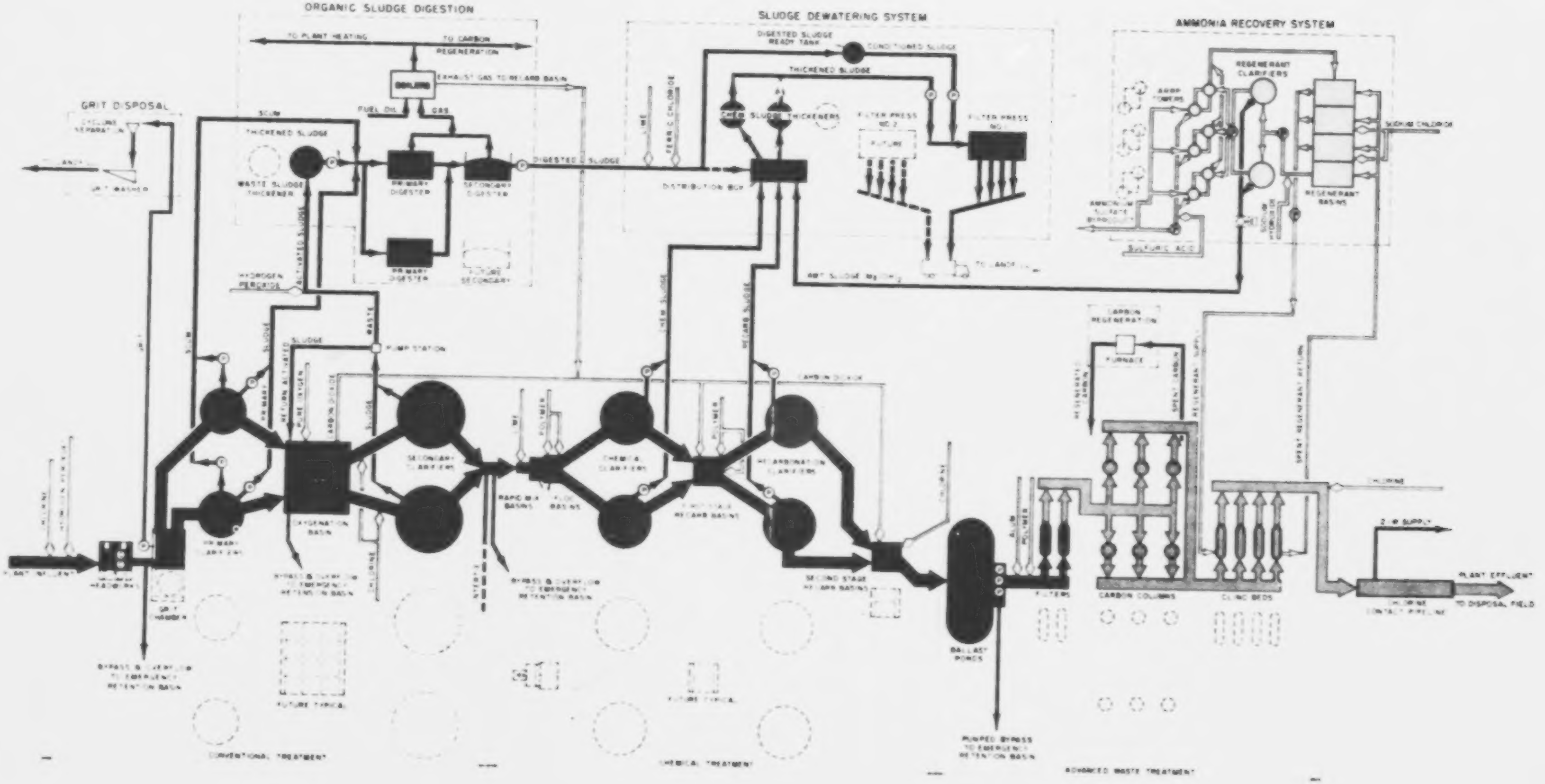
ORGANIC SLUDGE DIGESTION
TO PLANT HEATING
TO CARBON REGENERATION
BOILERS
EXHAUST GAS TO RECARB BASIN
FUEL OIL
GAS
SCUM
THICKENED SLUDGE
WASTE SLUDGE THICKENER
PRIMARY DIGESTER
SECONDARY DIGESTER
PRIMARY DIGESTER
FUTURE SECONDARY
DIGESTED SLUDGE
HYDROGEN PEROXIDE
ACTIVATED SLUDGE
GRIT DISPOSAL
CYCLONE SEPARATION
LANDFILL
GRIT WASHER
GRIT
SLUDGE DEWATERING SYSTEM
DIGESTED SLUDGE READY TANK
CONDITIONED SLUDGE
THICKENED SLUDGE
LIME
FERRIC CHLORIDE
CHEM SLUDGE THICKENERS
FILTER PRESS NO. 2
FUTURE
FILTER PRESS NO. 1
DISTRIBUTION BOX
TO LANDFILL
ART SLUDGE $Mg(OH)_2$
AMMONIA RECOVERY SYSTEM
AMM TOWERS
REGENERANT CLARIFIERS
REGENERANT BASINS
SODIUM CHLORIDE
AMMONIUM SULFATE BYPRODUCT
SULFURIC ACID
SODIUM HYDROXIDE
CARBON REGENERATION
FURNACE
REGENERATED CARBON
SPENT CARBON
REGENERANT SUPPLY
SPENT REGENERANT RETURN
PLANT INFLUENT
CHLORINE
HYDROGEN PEROXIDE
HEADWORKS
GRIT CHAMBER
BYPASS & OVERFLOW TO EMERGENCY RETENTION BASIN
PRIMARY CLARIFIERS
PRIMARY SLUDGE
RETURN ACTIVATED SLUDGE
PURE OXYGEN
CARBON DIOXIDE
WASTE
PUMP STATION
OXYGENATION BASIN
BYPASS & OVERFLOW TO EMERGENCY RETENTION BASIN
SLUDGE
CHLORINE
SECONDARY CLARIFIERS
NITRATE
BYPASS & OVERFLOW TO EMERGENCY RETENTION BASIN
LIME
POLYMER
RAPID MIX BASINS
FLOC BASINS
CHEMICAL CLARIFIERS
CHEM SLUDGE
CARBON DIOXIDE
POLYMER
FIRST STAGE RECARB BASINS
RECARBONATION CLARIFIERS
RECARB SLUDGE
SECOND STAGE RECARB BASINS
CHLORINE
BALLAST PONDS
PUMPED BYPASS TO EMERGENCY RETENTION BASIN
ALUM
POLYMER
FILTERS
CARBON COLUMNS
CLINO BEDS
CHLORINE
Cl_2 SUPPLY
CHLORINE CONTACT PIPELINE
PLANT EFFLUENT
TO DISPOSAL FIELD
FUTURE TYPICAL
FUTURE TYPICAL
CONVENTIONAL TREATMENT
CHEMICAL TREATMENT
ADVANCED WASTE TREATMENT

Table 1.

T-TSA Discharge Requirements

Constituent	Units	10-Sample Average	Daily Maximum
COD	mg/l	15	40
Suspended solids	mg/l	2.0	4.0
Turbidity	ntu	2.0	8.0
Total Nitrogen	mg/l	2.0	4.0
Total phosphorus	mg/l	0.15	0.4
MBAS	mg/l	0.15	0.4
Total dissolved solids	mg/l	440	—
Chloride	mg/l	110	—
Total coliform organisms	mpn/100 ml	—	23

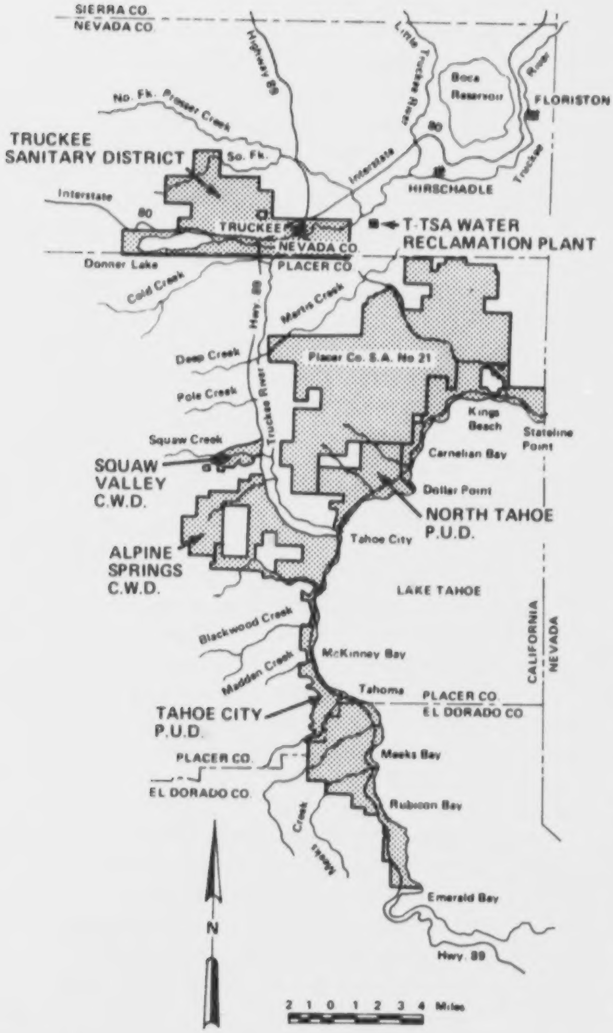
SIERRA CO
NEVADA CO
Highway 89
Little Truckee River
Boca Reservoir
River
FLORISTON
No. Fk. Prosser Creek
80
TRUCKEE SANITARY DISTRICT
So. Fk.
Interstate
Truckee
HIRSCHADLE
Interstate
80
TRUCKEE
T-TSA WATER RECLAMATION PLANT
NEVADA CO
PLACER CO
Donner Lake
Cold Creek
Martis Creek
Hwy. 89
Deep Creek
Placer Co. S.A. No 21
Pole Creek
Truckee River
Kings Beach
Stateline Point
Squaw Creek
SQUAW VALLEY C.W.D.
Carnelian Bay
Dollar Point
NORTH TAHOE P.U.D.
Tahoe City
ALPINE SPRINGS C.W.D.
LAKE TAHOE
Blackwood Creek
McKinney Bay
CALIFORNIA
NEVADA
Madden Creek
Tahoma
PLACER CO
EL DORADO CO
TAHOE CITY P.U.D.
PLACER CO.
EL DORADO CO.
Meeks Bay
Rubicon Bay
Creek
Meeks
Emerald Bay
N
Hwy. 89
2 1 0 1 2 3 4 Miles

Figure 2
T-TSA Service Area

at a total cost of $32 million. Full tertiary treatment, including maximum removals of nitrogen and phosphorus, was deemed necessary to protect the quality of the Truckee River into which the effluent would ultimately find its way after percolation into the permeable glacial outwash soil near the plant. This would ensure the safety and integrity of this primary water source for Reno, Nevada, a major city 48 km (30 miles) downstream.

Studies of the background concentration of pollutants in the Truckee River revealed that the critical pollutants were total nitrogen and total phosphorus. These were determined to be the most difficult contaminants to remove; other pollutants could easily be removed by proven processes. The primary goal of process design was to ensure that the treated effluent, when mixed with background concentrations existing in the river, would not violate the river standards even if discharged directly into the river. Percolation into the Martis Valley soils would provide an additional treatment factor where a residence time in the soil of about 150 days would be experienced, and the effluent's mixing with natural groundwater moving through the soil would result in additional dilution of its residential pollutants.

Startup

The facility was accepted for operation on February 1, 1978, with several major incomplete, faulty or untested systems including the secondary digester, the filter press and the ion exchange system. Sewage was accepted at that time on a limited basis from an interceptor connection from the Truckee Sanitary District in order to determine if the plant could be operated on a sustained basis. Flow from the one source was estimated to average 190 cu m/day (0.5 MGD).

Seed activated sludge was trucked from a nearby facility in Nevada operated by the Incline Village General Improvement District. A two-foot snow storm following the first day of sludge hauling prevented further addition of seed sludge to desired levels. Nevertheless, activated sludge did develop rapidly and reached an acceptable mixed liquor suspended solids concentration after two weeks of operation, even with the influent sewage temperature being in the range of 4 to 6°C.

Empty clarifiers during startup detained the passage of partially treated sewage through the plant. Use of the emergency retention basin (ERB), which has a capacity of 57,000 cu m (15 MG), further delayed startup of the advanced waste treatment (AWT) facilities and the processing of sewage through them for over three weeks. Potable water remaining in the plant from the contractor's systems checkout was processed through AWT prior to the passage of sewage and provided additional systems testing experience.

All sewage was comminuted and degritted and sent to aeration after primary clarification. All primary sludge was pumped to the oxygenation basins during the first several weeks in order to build up the activated sludge solids as fast as possible. Anticipated problems from the primary sludge were encountered but were offset by the advantage of the rapid buildup of activated sludge solids.

The partially treated sewage from the secondary system was given full chemical treatment with lime and recarbonation and pumped to the emergency retention basin to be reprocessed after the biological population had developed in the secondary system.

During the beginning of March an unexpected problem was encountered—a severe algae bloom in the emergency retention basin. Reprocessing of this water resulted in the only period of operation to date during which the final effluent suspended solids concentrations and turbidities were above discharge limits. No further bypasses to the ERB were permitted until it could be fully drained and dried. Subsequent use of the ERB has been restricted to bypassing only during major process failures or outages which have been infrequent.

Additional sewage flows from the remaining member districts were permitted into the plant beginning February 17. By March 1, the plant was accepting flow from all scheduled sources. Severe infiltration problems from snowmelt were detected during the latter part of February and continued through April as evidenced by high sewage flows (Figure 3.) and low sewage strengths. During Easter week, flows in excess of the design capacity of the plant were encountered with a peak seven day average flow of 18,000 cu m/day (4.8 MGD). This week, traditionally the last major ski week of the season, was unseasonably warm and resulted in coincident peak flows from the large tourist population and from heavy infiltration. Infiltration is a local district problem which each district must individually correct. Because of flow limits allocated by the State Water Resources Control Board to each district which must include accounting for infiltration, the two major districts located in the Tahoe Basin have enacted self-imposed building moratoriums on their districts until their infiltration problems can be substantially reduced

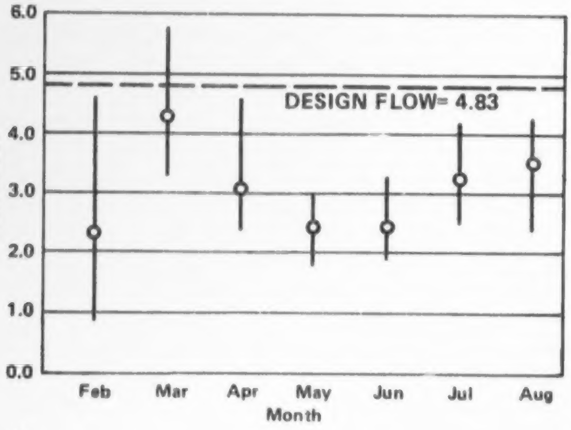

Figure 3
T-TSA Monthly Average Flow

Conventional Treatment

Following the development of a healthy activated sludge population, the secondary treatment system as well as the entire conventional treatment process have achieved their design objectives with the exception of two brief periods.

The first period occurred as the ski-tourist season ended and raw sewage loadings diminished. With all four oxygenation trains in service, aerator loadings reduced to less than 0.2 kg BOD/day/kg MLVSS (0.2 lbs. BOD/day/lbs. MLVSS) and resulted in over-aeration, frothing and secondary clarifier solids losses to a maximum of 57 mg/l. Removal of two oxygenation basin trains from service had the problem under control in less than one week. No nitrification was observed during the period nor during any period to date. Chemical treatment and advanced waste treatment effectively controlled the problems associated with the secondary solids carryover maintaining final effluent suspended solids and COD within discharge limits.

The second period was of slightly longer duration and resulted from a more severe problem. Failure of the filter press to dewater adequate quantities of anaerobically digested sludge caused massive recycling of digested solids to the primary clarifiers and at one point virtually eliminated the ability to waste activated sludge. Carryover of the digested sludge from the primary tanks with their high sludge blankets to the activated sludge system together with large quantities of solids in the secondary system saw the secondary effluent quality gradually deteriorate. A concerted effort was made to rectify the filter press problems and to operate the filter press dewatering digested sludge all available hours.

Outstanding dewatering performance by the filter press following repairs rapidly reduced the solids inventory in the conventional treatment system and enabled the process to once again achieve its design objectives. Also during this period, chemical treatment and advanced waste treatment processes were able to reduce the final effluent SS and COD below discharge limits.

Average performance of the conventional treatment process in terms of several major pollutants can be ascertained from Table 2. Secondary effluent nitrite and nitrate concentrations have never exceeded 0.1 mg/l. Activated sludge settling characteristics as measured by sludge volume index (SVI) typically have been in the range of 70 to 100 with a maximum SVI of 140 and a minimum of 60 being observed.

Incorporated into the design of the pure oxygen activated sludge process was a feature in which pure oxygen is added only in the first two stages of the three stage aeration system. In the third stage, atmospheric air is drawn in to allow carbon dioxide dissolved in the mixed liquor to be stripped and captured for utilization in the subsequent recarbonation processes. Experience to date has shown that during periods of low organic loading, bicarbonate alkalinity will increase or only show a slight reduction across the third stage. Only during the

Table 2

Plant Performance (6 month averages)

Process Stream	SS (mg/l)	COD (mg/l)	T.N. (mg/l)	T.P. (mg/l)
Raw	156	338	32	10.5
Primary eff.	120	—	34	8.7
Secondary eff.	16	57	28	4.9
Chem. clar. eff.	17	—	—	0.7
Filter eff.	2.8	26	—	—
Car. col. eff.	2.8	15	25	—
Final eff.	1.3	12	3.7	0.3
Discharge Requirement	2.0	15	2.0	0.15
Overall Removal (%)	99.2	96.4	88.4	97.1

Table 3.

Chemical Treatment Relationships (summer)

Process Stream	pH	Alkalinity (mg/l)	Calcium (mg/l)	Total P (mg/l)	Ortho P (mg/l)
Mixed liquor					
2nd stage	6.8	351	—	—	—
Effluent channel	7.2	240	—	—	—
Secondary Effluent	7.0	216	38	6.35	5.78
Chem. clar. eff.	11.1	305	85	0.80	0.05
Recarb. clar. eff.	10.3	223	51	0.64	0.53
Final eff.	7.3	191	31	0.24	0.21

height of the summer tourist season did significant alkalinity reductions of 100 mg/l or more occur (Table 3). Carbon dioxide analysis of the off-gases supported this finding. Because of the low carbon dioxide concentrations, the off-gases from this process have not been routinely utilized in recarbonation. Perhaps as sewage loadings increase in the future, this feature may become practical.

With respect to the Pressure Swing Adsorption (PSA) pure oxygen generation system, there have been no significant operation or maintenance problems.

Chemical Treatment

A wide range of lime dosages as controlled by pH set points from 10.8 to 12.0 and a variety of organic polymers have been tried in the plant as well as in extensive laboratory tests. Experience has indicated that removals of phosphorus (P) to a level below 0.2 mg/l as P in the chemical clarifiers is virtually impossible at any lime dosage, polymer concentration, or polymer type except at very low clarifier overflow rates, whereas pH set points of 11.0 to 11.2 consistently yield chemical clarifier effluent phosphorus concentrations of 1.0 mg/l or below, even at the highest overflow rates experienced with one of the two clarifiers out of service. This residual can readily be removed by 5-15 mg/l of alum applied ahead of the mixed media filters provided the pH of the water to the filter can be kept below 7.5. Presently, this mode of operation with a lime dosage pH set point of 11.0 to 11.2 is used along with alum treatment.

A pH set point of 10.8 or above has been found to be very effective in removing soluble ortho P. However, turbidity in the chemical clarifier effluent contains chemically precipitated phosphorus which readily reverts to soluble ortho P in recarbonation as the pH of the water is lowered (Table 3.). Increasing the pH set point above 10.8 results in less and less turbidity carryover with sparkling clear effluents produced above a pH set point of 11.4. As the pH set point is raised, however, more lime is used, more soluble calcium is carried over to recarbonation, and more turbidity is created through recarbonation. For the slight amount of additional phosphorus removal at a higher pH set point, the accelerated requirements for lime do not prove cost effective.

A variety of pH set points have been tried for control of the first stage recarbonation process. Reduction of soluble calcium (Ca) to a level of 40 mg/l as Ca has been the lowest achievable on a consistent basis. A pH set point of 10.1 to 1.04 has been required for this. Reduction of the pH set point below 10.0 has resulted in higher calcium concentrations as the result of the formation of soluble calcium bicarbonate.

The first stage recarbonation reaction which forms calcium carbonate appears to be time dependent. It has been consistently observed in the recarbonation clarifiers that the water in and near the clarifier center ring is clear and that a cloud of turbidity develops toward the outer periphery of the clarifiers. Consequently, polymer treatment in the recarbonation clarifier has been ineffective and has been abandoned.

Because of the formation of turbidity in the outer reaches of the clarifiers, considerable turbidity is carried over the weirs during high flow periods. Operation of the lime treatment system at pH set points of 11.0 to 11.2 minimizes the soluble calcium entering the recarbonation system, and, hence, minimizes the effects of turbidity losses from the recarbonation clarifiers.

Control of the pH in and following second stage recarbonation has been the plant's foremost problem. Addition of carbon dioxide in excess of several times the stoichiometric amounts during numerous peak flow periods has failed to reduce the effluent pH below 8.0. Several possible reasons postulated for this phenomenon relate to the low atmospheric pressure at the 5900 foot elevation of the plant; poor transfer efficiency in the basins; carbon dioxide stripping resulting from the high volumes of dilute boiler off-gases having 6-12% carbon dioxide content which are used; and to the low efficiency of the second disassociated hydrogen ion of carbonic acid.

Temporary facilities for adding concentrated sulfuric acid to the ballast ponds to reduce the AWT influent pH to 7.5 or below were installed during the middle of August. A permanent facility has been designed and will be installed in the near future. Since mineral acid addition was implemented and pH control to AWT attained, the effluent from the plant (for a continuous period of over 30 days at this writing) has not exceeded its discharge limits for SS, COD, Total P or Total N.

The importance of pH control to AWT is apparent when the following facts are considered:

- The effectiveness of alum in removing phosphorus is diminished considerably as the pH exceeds 7.5.
- The effectiveness of carbon adsorption in removing COD decreases with increasing pH.
- As pH increases more and more ammonium ion reverts to non-ionized ammonia which passes through the cation exchange media to the effluent unchanged. Additional ammonium ion previously taken up by the media can also be reverted to the non-ionized form and be rejected from the media.

The chemical treatment system as a whole has unquestionably been the most troublesome with respect to maintenance. Major problems have included:

- Frequent blockages in transfer lines in the dry lime system from lime fines or coarse rejects resulting in intermittent periods of failure to dose lime.
- Inefficient lime slaking requiring excess quantities (up to 50%) of lime to be dosed.
- Ineffective removal of grit from slakers causing severe abrasion of piping, valving and pumps and requiring replacement of valving, rotors and stators after less than 6 months operation; also forming deposits in clarifiers and thickeners which cause tank mechanisms to trip out from over-torque and which result in frequent blockages of sludge withdrawal lines.
- Formation of calcium carbonate scaling on clarifier launders and sludge pump impellers, and deposits of chemical sludge in connecting structures, recarbonation basins and ballast ponds.
- Insufficient pH control with carbon dioxide during peak flows.
- High costs due to large quantities of lime and liquid carbon dioxide required and to excessive maintenance problems.

Solids Handling System

Anaerobic Digesters. Since primary sludge solids were recycled to the aeration tanks during the first three weeks of startup, the startup of the digester system was initially delayed. Furthermore, as activated sludge solids built up, additional aeration trains were put on line delaying considerably the time before any waste activated sludge was available to be pumped to the digestion system.

One primary digester filled with water and heated to 32°C was put on line when primary sludge was first wasted. Intermittent feeding was used so that the contents could be recirculated after being continously fed for a period of time but allowed to settle before refeeding. Extensive rework to the floating cover of the secondary digester by the contractor prevented its use. Overflow from the primary digester was returned to the plant headworks. As the waste solids quantities increased, the second primary digester was put in service. Each digester was alternately fed.

Within several weeks, gas was being generated and flared. Shortly thereafter, the secondary digester became available and was put into service. Extensive problems with rag accumulations in the heat exchangers and with insufficient steam supply to the heat exchangers caused erratic temperature control and consequently intermittent periods of foaming and low pH. At one point lime addition for over one week was required before pH control could be regained.

Gas production and volatile solids reduction to date have been good.

Filter Press. The filter press was plagued by startup problems resulting almost entirely from the feed pumping system. Feed pump rotors and stators were found to be frozen to each other and were destroyed during initial testing. Problems were also encountered with the hydraulic drive system for the pumps. Chemical sludge dewatering was given priority whenever the filter press was usable as the chemical sludge would form an umpumpable paste if allowed to settle for any extended period of time. One problem followed another until three months after startup when the plant had a reliable system. Minimal amounts of digested sludge had been dewatered, and digested and raw sludges accumulated within the plant.

A further complication to removing dewatered solids from the plant was the fact that the solids were removed by a scavenger to a local landfill, precluding night pickup and seven day per week operation.

Recently, the filter press has worked quite well as indicated by the performance data presented in Table 4. Initial poor experiences with improperly conditioned digested sludge, necessitating time-consuming manual cleaning of the filter media, prompted the operators to devise a scheme to reduce this problem. Their present practice is to precoat the media with chemical sludge which dewaters readily without conditioning and then add the conditioned digested sludge. Even if the digested sludge is improperly conditioned, the cake will release and require little if any medial cleaning.

Table 4.

Filter Press Performance

	Chemical Sludge	Digested Sludge	Combined Sludge
Feed solids			
Range (%)	6-20	2.8-3.6	—
Dewatered solids			
Range (%)	52-66	39-52	43-54
Average (%)	58	46	49
Design (%)	55	35	—

Advanced Waste Treatment

Mixed Media Filtration. Removals of suspended material across the mixed media filters has been consistently good. Neither addition of alum nor polyelectrolyte to the filter influent have proven effective in either increasing the length of filter runs or in reducing filter effluent turbidity. Addition of either chemicals alone or in combination typically results in increased filter head loss and consequent shorter filter runs between backwash. Addition of alum has been required, however, for supplemental phosphorus removal. Average filter effluent SS and COD are presented in Table 2. Higher than expected SS concentrations have resulted from alum treatment of high pH water and from high alum dosages.

Activated Carbon Adsorption. The activated carbon adsorption column operation has been troublefree, and the removal process has worked consistently well in removing COD and MBAS. Indications of COD breakthrough were initially diagnosed as being attributable to exhausted carbon and the regeneration procedure initiated three times to date. During each regeneration procedure, 20% of the carbon in each carbon column was removed, regenerated and replaced. This was done contrary to the design recommendation of 10% carbon withdrawal because of problems in furnace operation during the first two regenerations which extended the regeneration process over long periods. During the third regeneration procedure, 20% carbon regeneration was again undertaken as the peak summer tourist season was approaching and because the regeneration process was performing so well.

In retrospect, it is now felt that the preliminary indications of COD breakthrough in carbon column operation were attributable to high pH water reducing carbon removal efficiency rather than to completely exhausted carbon columns.

Table 5.

Activated Carbon Adsorption, System Performance

Exhaustion Cycle	Average Contact Time (min.)	C.O.D. Final Eff. (mg/l)	C.O.D. Overall Removal (mg/l)	Carbon Usage (mg/l)	Carbon Usage (lbs/mg)	Performance (kg COD removed) / (kg carbon spent)
#2	47	11.3	13.2	52	431	0.26
#3	45	13.1	14.0	57	473	0.25

Because of the low frequency of regeneration, the scant data accumulated to date regarding carbon removal efficiency and carbon attrition, as presented in Table 5, are susceptible to considerable errors, especially when considering that twice as much carbon as recommended was regenerated each cycle and that the actual state of exhaustion was unknown prior to regeneration. Extensive carbon makeup during the first regeneration procedure due to unknown losses associated with the defining of the original carbon; the bedding of the furnace; and significant burning of carbon during the initial regenerations prevents use of preliminary data.

Average COD removals through the carbon columns for the first six months of operation are presented in Table 2. Location of the ion exchange beds with their fine-mesh media following the carbon columns has proven effective in reducing losses of carbon fines. Fines which represent both SS and COD are continuously washed out of the carbon colunmns in low concentrations and are washed out in high concentrations during the regeneration procedures.

Ion Exchange. In terms of achieving design objectives and in meeting effluent discharge requirements, the ion exchange system was the poorest performing system during the first five months of startup. The type of system employed has never been tried on a full scale, so considerable startup problems were anticipated. Additionally, this system was in the least-constructed state of the main stream process when the plant began operation, and further compounding the problem were the higher than anticipated ammonia loadings during the peak summer season and the pH control problems previously described.

Unquestionably, the ion exchange beds have been effective in removing ammonium ion to desired concentration levels as well as carbon fines. Recent data (Table 6), however, suggests that the exchange capacity is either below design requirements or has substantially deteriorated. This matter is presently under thorough examination. Had regeneration been able to keep pace with exhaustion, effluent standards could easily have been met with respect to total nitrogen

Realizing that the removal capacity of the ion exchange system was below actual loadings, supplemental ammonia removal in the form of breakpoint chlorination was employed beginning in mid summer. As the chlorine system was limited to a maximum dosage of 454 kg/day (1,000 lbs./day), the breakpoint chlorination procedure was used only when operator analysis of the clino bed effluent ammonia nitrogen showed chlorine requirements for total breakpoint chlorination to be within the limit of the system. Careful control of the clino system, coupled with extensive monitoring by

Table 6.

Ion Exchange Bed Performance (summer)

	Design	Actual
Flow to Exhaustion		
cu m/bed	6100	3000
(mil. gal./bed)	(1.62)	(0.8)
Time to regenerate		
hours/bed	6.95	7.8
Regeneration capability		
beds/day	3.45	3.0
Inf. NH_3-N		
mg/l	24.0	30.2
NH_3-N loading		
Kg/bed/cycle	127	91
(lb./bed/cycle)	(280)	(201)
Design flow loading		
Kg NH_3-N/day	439	552
(lb. NH_3-N/day	(966)	(1217)
Removal capability with all available beds		
Kg NH_3-N/day	439	281
(lb. NH_3-N/day)	(966)	(619)
Removal capability at design flow		
% NH_3-N removed	100	50.9

Table 7.

Ion Analysis to Determine Extraneous TDS Sources.

(samples composited over a period of one week during which sewage flow averaged 8300 cu m/day (2.2 mgd)

	Raw (mg/l)	Final (mg/l)
Cations		
Ammonium (as NH_4+)	27	.3
Calcium	25	37
Magnesium	3	2
Potassium	11	11
Sodium	50	278
Total cations	116	328
Anions		
Bicarbonate (as HCO_3-)	178	290
Chloride	50	252
Phosphate (as $PO_4{-}^3$)	8	0.08
Sulfate (as $SO_4{-}^2$)	25	55
Total anions	261	597
Sum of cations/anions	377	925
Tot. dis. solids	205	725

operators, has enabled the combination of ion exchange and breakpoint chlorination to reduce total nitrogen to within discharge limits. Recent low ammonia loadings on the clino system hve reduced the need for supplemental breakpoint chlorination.

Studies related to shortening regeneration cycles and multiple bed regeneration are presently underway to determine if the existing system can be modified to provide removal of design loadings without substantial capital improvements.

Problems limited to the exchange beds themselves have been few but significant. Butterfly valves which failed to properly seat in the high pressure system have caused either brine leakage to the effluent or process water leakage to the brine basins necessitating blowdown from the basins to restore proper levels. Inefficient purging of the brine from the beds following regeneration also has caused excessive brine losses.

Brine loss to the final effluent has been found to be related to frequency of regeneration rather than process flow. Since the required frequency of regeneration exceeds the design capacity of the system, regeneration is continuous, and brine loss is fairly consistent on a day to day basis. Consequently, as flow decreases, final effluent total dissolved solids (TDS) increases. A laboratory study using week long composite samples of raw sewage and final effluent during a period in which the sewage flow to the plant averaged 8,300 cum/day (2.2 MGD) showed that the TDS increases of final effluent over raw sewage result mainly from sodium, chloride, and bicarbonate ions (Table 7.).

No abnormal scale deposits have been found within the beds on structures or on the media. However, accumulations of scale have been found in the backwash line which also carries the final rinse waters after most of the brine has been purged.

The presence of high calcium concentrations and high pH in the brine and the presence of high bicarbonate alkalinity in the process water are conducive to calcium carbonate formation at any point where brine and process water interface.

The ammonia removal and regenerant recovery system has been plagued by bizarre problems, most noteworthy of which has been calcium carbonate formation in pipelines, pumps, spray nozzles and stripping media. Scale formation during the first few months was extremely severe and rapid, necessitating nozzle and pump impleller cleaning in hydrochloric acid as frequently as at two week intervals. Reducing free fall from the regenerant clarifier weirs by addition of standpipes in the discharge pipe and sealing of the regenerant clarifiers' coverplates to reduce absorption of carbon dioxide from the air have considerably reduced the scale problem. The effect of temperature on the solubility of calcium carbonate, however, has not yet been ruled out as a major factor in scale formation.

Most of the scale deposits on the media in the stripping towers occurred on the top few inches of the media. Twice the top few inches of media were removed for acid cleaning. On July 11, a temporary acid feed and recirculatin system was set up to acid clean the entire stripping system—piping, pumps, nozzles and media. Approximately 1,000 gallons of concentrated hydrochloric acid were added to the system before the neutralization reaction slowed down. A permanent acid wash system has been designed and will be installed in the near future.

A second significant but less complicated problem has also occurred in the absorber system. In the absorber system ammonia laden air from the stripping tower is blown through media being sprayed with an acidic solution. sulfuric acid in the solution is to absorb the ammonia as well as moisture to form an acidic solution of ammonium sulfate having a concentration below its saturation value. However, the absorption rate of water has been less than predicted resulting in severe crystalization of ammonium sulfate within the media, piping and pumps, reducing the efficiency of air flow or absorber recirculation and, hence, reducing overall removal of ammonia. Pursuant to discovery of this problem, operators have been instructed to monitor the specific gravity of the solution daily and to add dilution water when necessary to keep the solution slightly below saturation.

Table 8.

Regenerant Brine Characteristics (after 6 months operation)

	Concentration (mg/l)	(meq/l)
Cesium	0.004	neg.
Rubidium	0.005	neg.
Potassium	1400	36
Sodium	9250	402
Calcium	4000	200
Aluminum	1.5	0.2
Magnesium	9.2	0.8

Brine Strength — 2.3% as sodium chloride

Table 9.

Chemical Costs (6 month averages)

Process	Product	Dosage	Treatment Cost	
		(mg/l)	¢/cu m	¢/1,000 gal.
Boilers	Fuels	—	2.5	9.4
Phosphorus	Lime	449	3.3	12.7
Removal	Liquid CO_2	192	1.0	4.0
	Alum	24	0.3	1.0
($5.37/lb.	Polymer	1.4	0.8	2.9
P-removed)	Total	—	5.4	20.6
Ammonia	Salt	153	0.6	2.4
Removal	Caustic soda	153	3.1	11.7
	Sulfuric acid	91	.5	1.8
($0.95/lb.	Liq. chlorine	12.5	.3	1.1
N-removed)	Total	—	4.5	17.0

A recent analysis of the brine solution to determine the concentrations of impurities which build up after prolonged recycling is presented in Table 8. The cations are listed in the table in order of exchange preference by the clinoptilolite. The low concentrations of cesium and rubidium which are preferentially selected by clinoptilolite are indicative of their relative absence in the sewage. Buildup of calcium to high levels has been anticipated. Removal of calcium could be accomplished simply, but at considerable expense, so it is not practiced. The overall effects of the high calcium concentrations will, however, have to be studied in much greater detail.

Costs

Operation and maintenance costs for each unit process have not been determined because of the overall complexity of the treatment plant and the limited metering equipment from which to base cost. Additionally, the startup phase of operation required considerable experimentation which would not prove cost effective on a sustained basis.

Since the use of chemicals in the plant is so extensive and a major portion of the annual budget, average usage and resulting treatment costs of the high-use chemicals have been summarized in Table 9 with respect to phosphorus and ammonia removal. Boiler fuel costs have been included, since all flue gas is used as a source of carbon dioxide in the recarbonation process. Boilers are used only to supply heat and air conditioning for buildings and for maintaining mesophilic temperatures in the anaerobic digesters.

The dosages presented are average and probably considerably higher than optimum. As operation of the various chemical treatment processes as well as all other processes stabilizers, more meaningful costs will be generated.

Table 10.

Plant Performance (last 30 days)

	Flow		SS	COD	T.N.	T.P.
	(m^3/d)	(mgd)	(mg/l)	(mg/l)	(mg/l)	(mg/l)
Raw sewage	10,200	2.69	199	435	39.2	10.5
Final effluent	—	—	0.5	8.8	0.8	0.12
Discharge requirement	18,300	4.83	2.0	15.0	2.0	0.15
Overall removal (%)	—	—	99.8	98.0	98.0	98.9

Summary

The Tahoe-Truckee Sanitation Agency's advanced waste treatment plant has completed its startup phase of operation. Many operational problems were encountered and solved while others, which are more complex, are under study. Major problems affecting overall plant performance in meeting all discharge standards with the exception of TDS and chlorides due to brine losses are believed to have been brought under control. Recent plant performance (Table 10) has been exceptionally good, and it is anticipated that future performance will continue to be good and provide a product water exceeding the quality required by the plant's discharge requirements.

Tertiary Treatment of Biologically Purified Waste Waters Aimed at Their Recycling in the Production Processes.

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Moscow, "NIOPIK"
(Scientific Research Institute for
Organic Intermediates and Dyestuffs).

Rapidly developing branches of the USSR chemical industry resulting in the increasing amounts of waste waters polluted with different chemical products as well as the water quality standards getting more rigid and strict with every coming day make it necessary to take to various and new treatment techniques, some of which permit to reuse the treated waters in the production water cycle.

This paper describes the tertiary waste water treatment system at the "Pigment" enterprise in the city of Tambov. The waste waters are biologically pretreated. The authors present optimal after-treatment parameters; the treated waters can be recycled in the closed water supply system of the enterprise.

The tertiary waste water treatment at this enterprise comprises the following technological stages:

1) filtration,
2) desalination,
3) ozonization.

The total amount of waste waters including those from the shops already used today as well as those to be commissioned in the nearest future, all of them subject to biological treatment, will come to 5000 m^3/day.

All biologically pretreated waste waters, with their tertiary purification completed, can be recycled to the production processes.

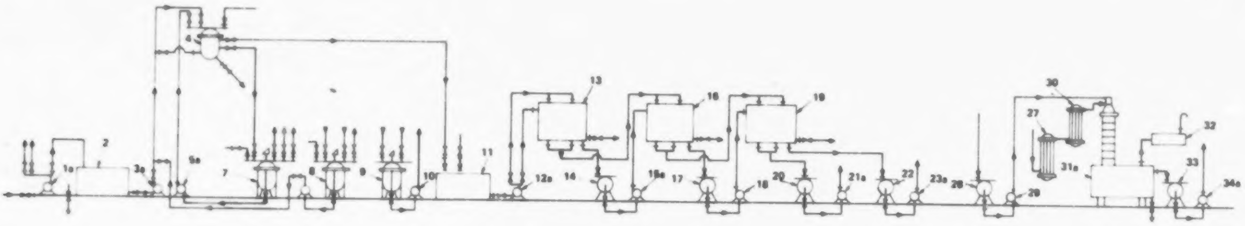
The tertiary treatment system receives waters from biological treatment facilities. These waters meet the following requirements:

	Actual Data	"VNIIVODGEO" Standards
Temperature, °C	up to 25	25-40
Suspended matter, mg/l	35-40	up to 20-30
Oils and resin-formers, mg/l	—	up to 10-20
Odour, in balls	up to 3	up to 3
pH	7.5-8.5	7.2-8.5
Hardness	6.9	7
Total salt content, mg/l	up to 1500	up to 1300
Chlorides, mg/l	105	150-300
Sulphates, mg/l	468	350-500
Permanganate oxidation, mg O_2/l	48-52	10-15
BOD_c	15-20	up to 15-20

It is obvious that after their biological treatment these waste waters do not meet the standards in the following parameters:

suspended matter,
total salt content,
permanganate oxidation.

To provide these waters with the qualities required by the official standards the enterprise uses the tertiary treatment system, including filtration, desalination and ozonization, upon which they are recycled to the production processes.



The flow diagram of this tertiary treatment system is shown in Fig. 1.

Filtration of Waste Waters Before Their Desalination

From the secondary biological treatment facilities the waste waters are pumped (following Scheme 1a, b) into a tank (Scheme 2) wherefrom they go to cartridge filters (following Schemes 3a, b and 4a, b), the filters are of the type PAP 80-248; they work on some auxiliary material such as filterperlite. Lavsan sleeve cloth of the type TG-519 is used as the filter partition. The filterperlite suspension is prepared upon the offered scheme (7). The volume of the suspenzator is 159% larger than that of the pipeline and the filter body. To make one layer of filterperlite for one operation it is necessary to take 40 kg of perlite as 0.5% suspension. Filtration is accomplished with a perlite as 0.5% suspension. Filtration is accomplished with a perlite feed of 100 g/m^3 of waste waters. The filtration efficiency is 500 l/m^2/h; the time necessary to make one layer is 0.5 h; the working cycle is 5 hours; residue disposal and filter regeneration take half an hour.

Perlite is fed to the suspendator pneumatically. The filterperlite suspension is directed to the filters by pumps (Scheme 5a, b). We use a 2% suspension prepared as described in Scheme 8 and pump it to the filter (Scheme 6a, b). After each cycle the filter is washed with hot water (70-80°C), about 3 m^3 per each cycle.

The sludge suspension from the residue disposal and filter regeneration is collected in a special collector (Scheme 9) wherefrom it is pumped to the sludge pond (Scheme 10).

The filtrate containing up to 5 mg/l suspended matter with particles of 1 mkm enters the filtrate tank (Scheme II).

Desalination

After filtration the waste waters containing up to 1.5 g/l salts are pumped to a dialyzer (Schemes 12a, b and 13). The electrodialyzer is made of equal numbers of deionization and concentration chambers which make a sequence of packets balt-tightened with one another. The upper and lower parts of the packets contain electrodes through which some direct current voltage is supplied. Heterogenic membranes MK-40 and MA-40 have been used. Their characteristics is as follows:

	MK-40	MA-40
Swelling capacity, %		
in thickness	134.4	127.7
in area	121.1	121.0
S.V.C. mg-equi/g, dry membrane	3.03	3.56
Volume resistivity, ohm/cm	210.3	258.8
Selectivity	0.9	0.82
Tensile strength, kg/cm^2	183.5	195.2

The waste waters enter the electrodialyzer by three independent tracts, one of them is for washing the electrode chambers (of cathode and of anode), the second is for desalted water; here the waters are purified from ions which, following their own charge, are directed either to the cathode or to the anode; the third one is for concentration; here the solution is saturated with ions.

The desalted waters contain up to 0.5 g/l of salts which corresponds to the total salt content in river waters used for water-rotation cycle. These waters enter a tank for storage (Scheme 28).

The solution contains up to 2.5 g/l of salts when entering the tank (Scheme 14) from which it is pumped (Scheme 15a, b) to the electrodialyzer (Scheme 16). The desalted waters contain up to 0.5 g/l of salts and enter tank N28. The liquid from Device N16 contains up to 4.5 g/l of salts when it goes to the tank (Scheme 17) wherefrom it is pumped to the electrodialyzer (Scheme 19). Leaving this device, the water contains up to 0.5 g/l of salts and goes to tank N28 while the solution with about 8 g/l of salts enters tank N20 from where (Scheme 21a, b) it is disposed into deep well-isolated horizons; in total, some 12.5% of all waters are disposed of in this way after desalination.

In all the described electrodialyzers (Schemes 13, 16, 19) during desalination the electrodes are washed in a closed cycle. The filtered waters from the tank (Scheme 11) are pumped (Scheme 12a, b) for washing the electrodes of the dialyzer (Scheme 13), after which the electrode chambers of dialyzers NN 16 and 19 are washed one by one. Then the washing waters are collected in tank N22 and pumped back to tank N11 (Scheme 23a, b).

The membrane life in the electrodialyzers is about 5 years on the average. With this time expired, the membranes are either partially or completely replaced.

Ozonization

Upon filtration and desalination, the waters from tank N28 are pumped (Scheme 29a, b) to the mixing chamber of the contacting device (N31a, b) which is a rectangular horizontal tank with a vertical plate-like sectional column.

Before their ozonization the waste waters had the following main parameters:
1) suspended matter, mg/l — up to 0.5
2) total salt content, g/l — up to 0.5
3) permanganate oxidation, mg O_2/l — up to 48-52
4) colourity, in grades — 900.

To obtain ozon an ozon generator was used (OP-121) (Scheme 30) with efficiency of 1.6 kg/h (Scheme 30). The air coming to the ozonator after the frying device of the type UOVB-0.5M (Scheme 27) has the dew point equal to -50°C.

The ozon-air mixture with 15 g/m^3 of ozon is supplied to the mixing chamber of the contacting device (Scheme 31a, b) where simultaneously come the waste waters from device N28. The unreacted ozon from the contacting device (31a, b) enters the degasator (Scheme 32) filled with a catalyst and equipped with an electric air heater. Upon degasation, the ozon-free air is discharged in the atmosphere. After ozonization the waste waters overflow into a tank (Scheme 33a, b) and therefrom are pumped (Scheme 34a, b, c, d) back into the water rotation cycle.

Upon ozonization, the waste waters meet the following basic requirements:
1) suspended matter, mg/l — up to 5
2) total salt content, g/l — up to 0.5
3) permanganate oxidation, mg O_2/l — up to 15
4) colourity, in grades — up to 80.

Conclusions

The authors show that the waste waters from the "Pigment" enterprise in Tambov were biologically purified and after-treated on a tertiary treatment system working on the following technological scheme: filtration, desalination, ozonization; upon such treatment the waste waters were in complete agreement with the standards set up for water recycling. The desalinized waters, free of suspended matter and organics, return to the closed water rotation cycle of the enterprise.

The pickle containing up to 8 g/l of salts is disposed of into deep, safely isolated horizons; in total, about 10 to 12% of the total water amount entering the tertiary treatment system are disposed of in this way.

Legend to Fig. 1. Tertiary Waste Water Treatment

1a, b) pump to supply the waste waters; 2) tank for storing the waste waters; 3a, b) pump to feed the waste waters to the filter; 4) cartridge filter, type PAP 80-248; 5a, b) pump to supply the filterperlite; 6a, b) proportioning pump; 7) suspenzator to supply the perlite; 9) sludge collector; 10) pump to supply the sludge; 11) tank for the filtrate; 12a, b) pump to feed the filtered waste waters; 13) electric dialyzer, type EXO-5000 × 200; 14) receiving tank; 15) centrifugal pump; 16) electric dialyzer EDU-1-400 × 4; 17) receiving tank; 18a, b) centrifugal pump; 19) electric dialyzer EDU-1-400 × 4; 20) receiving tank; 21a, b) centrifugal pump; 22) receiving tank; 23a, b) centrifugal pump; 27) air drying device, type UOVB-9.5M; 28) tank; 29) centrifugal pump; 30) ozonator, type OP-12 B 1; 31a, b) contacting device; 32) degasator; 33a, b) tank for treated waste waters; 34a, b, c, d) centrifugal pump.

Protocol

Protocol of the 7th Meeting of the USA and USSR delegations on the problem of Prevention of Water Pollution from Industrial and Municipal Sources (Moscow, USSR, November 12-23, 1978).

In accordance with the Memorandum of the Meeting of the Joint USA-USSR Commission on Coorperation in the field of Environmental Protection (Moscow, November, 1976) the USA and USSR delegations held a meeting on problems of wastewater treatment in Moscow between November 12-23, 1978.

The American delegation was headed by Mr. Harold P. Cahill, Jr., Director of the Municipal Construction Division, U.S. Environmental Protection Agency.

The Soviet Delegation was headed by Prof. S. V. Yakovlev, Director of VNII VODGEO, Gosstroy USSR.

The list of participants is attached in Appendix I.

In the course of the meeting the following was accomplished:

1. A Symposium was held on the subject of "Advanced Treatment of Biologically Treated Effluents, Including Nutrients Removal".
2. The accomplishments of the 1978 Program of Cooperation were discussed.
3. The Working Program for 1979 was coordinated.

In the course of the Symposium 12 professional papers were delivered: 6 papers from the US side and 6 papers from the Soviet side. The titles of these papers are attached in Appendix II.

Of particular interest were the papers of American experts of ozone treatment and advanced treatment facilities, including those operating in the area of Lake Tahoe and papers of Soviet specialists on wastewater treatment on sand filters.

The delegations have agreed that each side will publish all reports in the necessary number of copies in its own language prior to Febrruray 1, 1979, and will distribute them to interested organizations. The sides will exchange 10 copies each of the published Proceedings of the Symposium.

2.

In the course of the meeting the specialists discussed the results of current research carried out in accordance with the Program of Cooperation and exchange scientific and technical literature. The delegations expressed their satisfaction that a 30-day exchange of specialists in the field of wastewater treatment and sludge handling was accomplished in 1978. Two American environmental engineers visited the USSR in July and two Soviet specialists visited the USA in November.

The exchange of specialists was of great scientific and practical importance. The sides noted the value of such exchanges for the future.

3.

The delegations agreed that in January 1979 they will exchange lists of selected projects for advanced treatment of waste waters. During 1979 the sides will exchange detailed design plans and specifications for these projects. This exchange will involve no cost to either sides.

The delegations defined and coordinated the Program of Cooperation for 1979 (Appendix III).

The sides have agreed that a Symposium on subject of "Treatment of Oil-Containing Wastewaters and also summary of experience in operating facilities for biological treatment of wastewaters using pure oxygen" will be held in the USA between April 8-20, 1979.

The following has been agreed upon for the preparation of the forthcoming Symposium:

- each side will present 5 professional papers for the Symposium;

- both sides will exchange the titles of papers to be delivered by February 1, 1979;

- both side will exchange the texts of the papers, both in Russian and English, at the beginning of the Symposium.

In the course of the Meeting in the USSR (Moscow, August 12-24, 1979) the delegations will continue to discuss and coordinate the details of the projects of experimental units for wastewater treatment and sludge handling.

In the course of the meeting the delegations will hear professional papers on the methods of wastewater treatment, facilities, units and equipment for experimental treatment plants.

The American delegation during this visit to the USSR was given a briefing on the operation of municipal and industrial waste waters treatment plants in Moscow, Novopolotsk, Togliati and visited a number of research organizations in Tallin, Leningrad, Moscow.

The sides expresssed their satisfication that the Meeting was held on a high scientific and technical level in an atmosphere of friendship and mutual understanding thus contributing to further development and strengthening of cooperation in the field of environmental protection.

This Protocol was signed on November 24, 1978, in two copies, in Russian and English, both texts being equally authentic.

From the US Side

Harold P. Cahill, Jr.
Head of Delegation

From the Soviet Side

S.V. Yakovlev
Head of Delegation

Appendix 1

List Of Participants at the Symposium "Advanced Treatment of Biologically Treated Effluents, Including Nutrients Removal"

From the American Side

Harold P. Cahill, Jr.	Delegation Leader Director, Municipal Construction Division, US EPA
Andrew Paretti	Consultant, US EPA
Frank P. Sebastian	Senior Vice President, ENVIROTECH Menlo Park, California, USA
Ossian R. Butterfield	General Manager and Chief Engineer, Tahoe-Truckee Sanitary Agency
Prof. Alexey N. Malyshev	Interpreter, US Department of State

From the Soviet Side

S.V. Yakovlev	Delegation Leader Director, VNII VODGEO, Gosstroy USSR
R.F. Slavolyubov	Department Chief of Main Administration, Gosstroi USSR
V.N. Shvetsov	Deputy Director, VNII VODGEO, Gosstroi USSR
I.N. Myasnikov	Laboratory Head, VNII VODGEO, Gosstroi, USSR
N.V. Pisanko	Chief Engineer, State Project Institute "Ukrvodokanal Project", Gosstroi USSR
E.A. Lobacheva	Senior Interpreter, VNII VODGEO, Gosstroi USSR

Appendix 2

List Of Papers Presented at the Symposium "Advanced Treatment of Biologically Treated Effluents, Including Nutrients Removal"

From the American Side

1. *Land Treatment:* Achieving Nutrient Removal by Recycling Resources — Thomas C. Jorling and Richard E. Thomas
2. *Operation of a 5 MGD Model Advanced Wastewater Treatment Plant at Piscataway, Maryland* — Harold P. Cahill, Jr. and Thomas P. O'Farrell
3. *Granular Carbon and Other Tertiary Treatment Processes* — Frank Sebastian and Dennis Lachtman
4. *Facilities for the Treatment of Biologically Treated Effluents from Industrial and Municipal Source by Ozone* — William Lacy.
5. *Municipal Wastewater Treatment by Land Application in the Minneapolis — St. Paul, Minnesota Area* — Richard Stanley and Douglas Wallace, Stanley Consultants Inc.
6. *Tahoe-Truckee Water Reclamation Plant* — Ossian Butterfield

From the Soviet Side

1. *Works for Final Purification of Municipal Wastes* — L.I. Gyunte and E.S. Razumovsky
2. *Aerated Filters for Tertiary Treatment of Secondary Effluents* — A.G. Kirichenko
3. *After treatment of Biochemically Treated Wastewaters Before Their Reuse* — E.G. Ioakimis and E.A. Yusupov
4. *Investigation of the Loding Regeneration of the Mixed-Media Filter with the Descending Particle Seize Distribution for the Advanced Wastewater Treatment* — Yu.N. Golovenkov, N.V. Kravtsova, A.S. Slavinsky, R.S. Chabirov
5. *The Mathematical Model of the Adsorption of Organic Impurities from Water Solutions with Microporous Adsorbents* — A.M. Stadnik
6. *Tertiary Treatment of Biologically Purified Waste Waters Aimed at Their Recycling in the Production Processes* — Yu.M. Latyshev and V.I. Zavarzin

USA-USSR Cooperation of Working Group on Prevention of Water Pollution from Industrial and Municipal Sources for 1979

NO	TITLES	FORM OF WORK	RESPONSIBLE FOR		TIME	EXPECTED RESULTS
			From the USSR	From the USA		
1	2	3	4	5	6	7
1.	Modernization of existing and development of new combined facilities with high efficiency for wastewater treatment, including hydrocyclones, multistage settlers, flotators, investigation of usage of flocculants and coagulants	Joint development of themes, scientific information and specialists delegation exchange. Symposium of: "Treatment of Oil-Containing Wastewaters and also summary of experience in operating facilities for biological treatment of wastewaters using pure oxygen" (USA, April 8-20, 1979) Visit of 6 Soviet specialists to the USA Visit of 6 American experts to the USSR, August 12-24, 1979	VNII VODGEO	EPA	1980	Improvement of the efficiency of existing and development of new treatment facilities, reduction of reagents and cost of wastewater treatment.
	Development of hydrocyclones and pressure flotation units		VNII VODGEO Gosstroy USSR		1980	Recommendations for designing hydrocyclones and pressure flotation units.
	Development of tubular and plate settlers			EPA	1980	Recommendations for use of settlers for wastewater treatment.
	Development of open aeration tanks of "Marox" type with the usage of technical oxygen			EPA	1979	Development of open aeration tank with the usage of technical oxygen.
	Development of closed combined aeration tanks of "Oxitank" type with the usage of oxygen.			EPA	1979	Development of closed aeration tanks with the usage of technical oxygen.
	Development of multimedia filters with gradually descending particle size distribution		VNII VODGEO Gosstroy USSR		1979	Recommendations for designing filters for treatment and final treatment of wastewaters.
	Development of multimedia filters and facilities with continuous washing.			EPA	1979	Recommendations for construction of multimedia filters.
2.	Intensification of wastewater treatment processes in petrochemical, chemical petroleum refining and pulp and paper industries	Information and delegation exchange	VNII VODGEO Gosstroy USSR	EPA	1979	Increasing of wastewater treatment efficiency of existing treatment plants, introduction of new treatment schemes, maximum usage of treated effluents in recirculating systems.
	Intensification of wastewater treatment processes in petroleum refining industry		VNII VODGEO		1979	Development of treatment scheme of a petroleum refining plant using mechanical, physical and biochemical methods.
3.	Development of highly efficient methods and facilities for municipal sewage treatment with removal of biogenous elements; usage of treated effluents in recycling systems at industrial enterprises.	Joint development of themes, information and delegation exchange	VNII VODGEO	EPA	1980	Development of new treatment facilities for prevention of water basing eutrophication; development of new treatment systems with maximum usage of treated effluents in recycling systems at industrial enterprises.
	Development of methods and facilities for nitrates and nitrites removal.		VNII VODGEO			Recommendations for designing units.
	Development of optimal schemes of facilities for nutrients removal			EPA		Recommendations for designing facilities

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			From the USSR	From the USA		
1	2	3	4	5	6	7
4.	Wastewater sludge handling	Information and delegations exchange	VNII VODGEO	EPA	1980	Reduction of cost of wastewater sludge handling, improvement of the efficiency of operation of sludge handling facilities.
	Wastewater sludge stabilization and dewatering processes		VNII VODGEO			Recommendations for designing facilities for sludge stabilization and dewatering
	Methods and facilities for sludge heat treatment and utilization			EPA		Recommendations for designing sludge heat treatment and utilization facilities.